Sewage Purification and Disposal

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"Principles and Practice of Plumbing," "History of Sanitation," "Plumbing Plans and Specifications," "Wrought Pipe Drainage Systems"

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GENERAL

Publisher's Note

THE primary object of our organization is, as is universally known, to manufacture and market "Standard" Plumbing Fixtures, Brass Goods and other products made in our factories. In the development of an organization to accomplish this result, there has been established an Advertising and Publishing Department of no small proportions, and "Sewage Purification and Disposal" is simply the outgrowth of the work of this department. This brief statement will, we believe, serve to give the public a clear understanding of our somewhat unique position of being at the same time manufacturers and publishers.

The first serious work of the Publishing Department on a large scale was "Modern Sanitation" (established June, 1904). From this came the publication, first in serial form and later as a book, of J. J. Cosgrove's first work, "Principles and Practice of Plumbing" (book published December, 1906). The phenomenal success of the book is a matter of general knowledge, although it may not be widely known that "Principles and Practice of Plumbing" has been adopted as a text book in more than thirty universities and colleges in the United States, and bids fair to be adopted in others. This magnificent achievement has been accomplished solely on the merit of the work and without solicitation on the part of either author or publisher.

There is now offered almost simultaneously two new books by Mr. Cosgrove, one being the volume in hand and the other "History of Sanitation."

In "Sewage Purification and Disposal," "Principles and Practice of Plumbing" and "History of Sanitation" we feel that the literature of the craft has been enriched in an enduring manner, and that we have fully justified our appearance in the field of publishers as amply as we have our standing as manufacturers of a world-wide known and used product.

Standard Sanitary Mfg. Co.

Pittsburgh, U. S. A.

Publishing Department

Preface

THE purification of sewage has been a growing problem in thickly populated centers ever since the danger to public health, arising from polluted waters, was realized. During that period numerous experiments on a large scale were conducted, and considerable data were worked out in practice, so that at the present time the principles of sewage purification are so well established that an intelligent engineer, grounded in the principles of the science, can construct works to effect any desired degree of purification of the crude sewage.

Up to the present time, however, no text book or treatise has been published on the subject giving in concise, ready form, rules, tables and data for designing and proportioning purification works. Further, the principles and data worked out by experiment and experience are scattered through widely separated reports, public documents and private papers, so that they are not in available form. For these reasons, and owing to the present active interest in the subject, it is hoped that SEWAGE PURIFICATION AND DISPOSAL will fill a want in the field of engineering literature, and be a guide to communities grappling with the problem.

The aim of the author in preparing the manuscript was to present, as simply as possible, a work which would show the method of constructing various types of sewage purification plants, their details and proportions, together with a description of materials best suited to the purpose, so that any one trained in engineering design, by following the text, can successfully plan and proportion a sewage disposal works. To this end the illustrations were purposely made simple, so as to show the principles of construction without being clouded by the numerous details of a large complicated plant. Perspective drawings were used in preference to mechanical drawings, so as to convey to the mind a true mental picture of the plant, or detail under discussion. It was assumed by the author that the designer would possess the necessary skill as a draftsman to make plans, and would simply want to know the size, shape and materials of the several parts.

The drawings for this volume are all original. No catalogue or other borrowed plates were used, and every effort was made to insert an illustration where it was thought that a drawing would help make clear the text.

By an avoidance of technical terms and the presentation of the text in a simple manner, it was hoped to make Sewage Purification and Disposal valuable, not only in the class room and for engineers and architects, but likewise to engineering and architectural students, municipal officials, sewage disposal committees, and everybody else interested in the subject.

J. J. Cosgrove

Philadelphia, Pa.,
March 15, 1909

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PRINCIPLES OF SEWAGE PURIFICATION

COMPOSITION OF SEWAGE

Conservation of Matter—In the economy of nature, nothing is destroyed. The molecules of matter are constantly undergoing changes that build up and destroy, and tear down to reassemble in other combinations; but in the changes that take place nothing is lost; the molecules or elements are simply grouped in other forms. In the cycle of changes that take place, inorganic matter is converted into plant life, plant life into animal tissue, and animal tissue back again to inorganic mineral matter, ready to begin anew the endless cycle. Vegetation, in growing, absorbs from the soil nitrogen in the form of nitrates. From the air the chlorophyl of the leaves and other green parts decomposes carbon dioxide, giving off the free oxygen and assimilating the carbon, which as cellulose enters into the composition of the woody tissue. The oxygen liberated by vegetation supports animal life, while the vegetable substance built up from inorganic compounds either dies or is used as food for animals, and so in turn becomes part of animal tissue.

Dead vegetable and animal matter, together with the matter excreted by animals during life marks another stage in the cycle of changes. Dead organic matter cannot be used as food by higher forms of vegetation. It contains all of the material necessary to vegetable life, but not in suitable form. The organic matter must first be worked over by lower forms of life, which liquefy the solids, release the carbon dioxide and convert the nitrogenous material into nitrates. This function is performed by *fungi*, *yeasts*, *bacteria* and *moulds*, which, having no chlorophyl with which to decompose from the carbon dioxide of the air the cellulose forming carbon, derive the cellulose which they contain, as well as all the substances by which they are nourished, from organic matter, either living or dead. They subsist like animals, by devouring plants or other animals; not like higher plants, which derive their nutriment from the soil and air. When this class of organisms have completed their work, dead organic matter has been reduced to inorganic compounds suitable for plant life, and the cycle of change is complete.

That is the process going on in nature at all times, while in the purification of sewage the last stage of the process, that which reduces the organic to inorganic, is carried on under artificial conditions that are the most favorable for the purifying organisms.

A sewage purification plant is primarily a bacterial farm, and to successfully design and operate one requires a general knowledge of the composition and decomposition of sewage, together with a thorough knowledge of the kind, functions and habits or requirements of the invisible vegetation to be cultivated. It is upon their multiplication, species and activity that the success of the plant depends, and to promote such growth, natural selection and activity, the conditions of light, air, food, temperature, dilution and space they require, under different conditions, must be known and provision made for these requirements.

Organic Matter—In its broadest sense, organic matter is anything that possesses life, or is the product of the vital activity of anything that possesses life. It might be vegetable matter like potatoes, rice or flax; animal matter like flesh, blood and bones; the product of vegetable life like the sap of a maple tree, or the product of the vital activity of animals in the form of milk, urine and excreta. Organic matter may be either living or dead. Life is intra-cellular activity; death, cessation of activity. In the living condition, organic matter is active. It possesses intra-cellular motion, together with certain vital resistance, that protects it from injury from without. After death both the motion and the resistance cease, and the cells that compose the organic matter become food for lower organisms, which reduce them to useful compounds.

In the fluid waste that makes up a town's sewage, organic matter is present in considerable quantity, both as vegetable matter and as animal matter, living and dead. It is only the dead organic matter, however, that gives to sewage its characteristic appearance and odor. This matter is very unstable and is constantly undergoing changes that break it down into simple compounds, usually with an evolution of foul-smelling gases. As a rule the quantity of organic matter in average sewage does not exceed one part in 1,000, yet it is this small portion of the whole that makes sewage objectionable, and it is the object of sewage purification works to oxidize into useful nitrate that one part of organic matter.

It is commonly supposed that urine and excreta are the two matters that make sewage objectionable. As a matter of fact, however, they contribute only their portion to the general result. If the discharges from water closets and urinals were excluded from the house drains, the composition of the sewage would be so little altered that, to the senses, the change would not be perceptible. However, while urine and excreta add but little to the objectionable appearance and odor of sewage, they are objectionable for sanitary reasons, as under certain conditions they might infect a water supply. For instance, when an individual suffers from a bacterial disease of the bowels, such as typhoid fever or cholera, his discharges contain the specific bacteria of that disease and might infect the sewage and thence find its way to some water supply, thereby causing an epidemic of the disease.

The organic matter of household sewage, outside of excreta and urine, consists of tea and tea leaves; coffee and coffee grounds; soap suds from washing, scrubbing, laving and scouring; finely divided organic matter removed with soap suds; grease from cooking and from the cleaning of cooking utensils; crumbs of bread and portions of other foods carried into the drain during the process of washing dishes; paper from water closets; milk, juices of fruits and vegetables extracted during the process of cleaning, preparing and cooking, and numerous other matters that enter into the composition of the household wastes.

The sewage from manufacturing towns is more complex than that from resident cities. In the former case many by-products of the various industries are added to the usual household waste. These by-products so change the character of the sewage that, to a greater or less extent, to produce satisfactory results it requires modifications in the construction and operation of the sewage purification works. Some of the industrial wastes that must be cared for by purification plants are: blood, hair, portions of animal tissue and the contents of animal intestines discharged into sewers by slaughter houses; malt and grain from breweries; woody fibers from paper mills; hair, wool, cotton and felt from cotton mills, wool scouring, felting and textile works; drainage from stables, and portions of vegetable matter from canning and preserving factories. In addition to the foregoing organic waste, some industrial concerns discharge into the sewer liquid wastes that are so strong that they greatly interfere with the ordinary biological process of decomposition. Among such liquid wastes may be mentioned dyes from dyeing works; chemicals from bleaching works; acids from tanneries, and galvanizing pickle from galvanizing plants.

Sewage purification plants in cities that have the combined system of sewers, that is, a system in which both rain water and sewage are conveyed, have imposed upon them the further burden of purifying the washing from street surfaces, yards, roofs and areas. This in itself is no inconsiderable burden, as it necessitates the reduction of most of the organic matter, such as droppings from horses, that litter the streets before a rain storm.

The living organic matter in sewage consists chiefly of the micro-organisms that bring about the change in the composition of the dead organic matter. These multiply rapidly when food is plentiful and the conditions of air and temperature are suitable, but they die quickly when the environment becomes unsuitable for their wants, and are soon reduced to inorganic compounds by the surviving organisms.

Nitrogenous Matter—Nitrogenous matter is widely distributed in nature, and in some form or other enters into the composition of all organic matter, both vegetable and animal. The source of all nitrogenous matter is the atmosphere, where, as nitrogen, it composes the greater bulk of the air.* In its gaseous state, nitrogen cannot be assimilated by animal and vegetable organisms. For the use of vegetation nitrogen must first be reduced to soluble nitrates,† while animal and lower forms of vegetable life, like fungi, moulds and bacteria, obtain their nitrogen from the proteids and albumin built up by chlorophyl bearing plants or by other animals.

During the time that nitrogen is a constituent of organic life, either vegetable or animal, it is a stable, insoluble substance; when the organic matter is dead, however, the nitrogenous substance becomes soluble and unstable, and is quickly attacked by putrefactive micro-organisms, which reduce it to more simple compounds. Nitrogenous compounds are what give to the organic matter in sewage its most objectionable quality.

The principal class of nitrogenous compounds entering into the composition of animal and vegetable organism is *albumin*. This is such a common constituent of organic matter in sewage that the amount found by chemical analysis is taken as an index of the quantity of organic matter in the sewage. A familiar example of albuminoid matter is the transparent albumen that forms what is known as the

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* Air, by volume; .21 oxygen, .78 nitrogen and .1 argon.

† Most vegetation obtain their nitrogen from organic matter that has been reduced to nitrates by nitrifying bacteria. Some plants, however, particularly those belonging to the leguminous group, obtain their supply of nitrates from nitrogen fixing bacteria that grow in nodules on the roots of the plants, and obtain their nitrogen direct from the atmosphere.

white of eggs. Urine, also rich in albumin, is another familiar example. In vegetable organisms albumin is present in the fluids and solids of the plant, while the fruit of leguminous plants, like beans, lentils and peas, also the meat of nuts, are rich in nitrogenous materials. The different forms of nitrogenous materials—albumin, protein, globulin, etc.—are all composed of the same substance, but contain varying proportions of the elements—carbon, hydrogen, oxygen, nitrogen and sulphur—of which all nitrogenous compounds are formed.

Carbohydrates—Vegetable organisms, composed of carbon, hydrogen and oxygen, in any of the various proportions that these gases may be grouped to form stable compounds, are known as *carbohydrates.* One of the chief constituents of carbohydrates is starch, or its derivative, sugar. Familiar examples of carbohydrates are sugar-cane, glucose and grapes.

Carbohydrates are always of vegetable origin. Their presence, therefore, is not indicative of animal pollution, while nitrogenous substances might be either of vegetable or of animal origin.

Cellulose—Cellulose is a carbohydrate, to the usual molecules of which is added one atom of nitrogen. Cellulose is interesting in sewage purification, principally from the fact that it is the chief constituent in the composition of the woody fiber that makes up the stalk, branches and leaves of plants. Filter paper, bleached cotton and cocoa-nut fiber are nearly pure cellulose.

Inorganic Matter—The inorganic matter in sewage usually is in solution, and consists principally of mineral salts that have been dissolved by the water when percolating through the earth or running along its surface. Sometimes inorganic matter is in suspension in finely divided particles like the minute flakes of clay that cause the turbidity of some rivers. Most of the inorganic matter in sewage is present as a constituent of the water that supplies the community, or that has leaked into the sewer from the ground water. The most common forms of inorganic matter in solution in sewage are acid wastes from factories, carbonates of lime, sulphates of lime, ferrous oxide, salt, in the form of chlorides, chiefly of sodium, with less quantities of potassium and ammonium, and nitrates. In suspension there is present more or less sand, grit, clay, silt and coarse particles in the form of pins, hair-pins and other metallic substances that have been accidentally introduced into the drainage system.

The process of sewage purification is less affected by the presence of coarse metallic particles than by the matter in solution. Inorganic matter in suspension can easily be removed by precipitation in a grit chamber, or it can be caught on the rough screen that usually protects the raw sewage inlet to purification works. If it passes the grit chamber and screen, upon entering the tank it settles in the sludge at the bottom and causes no trouble. Soluble impurities on the other hand, affect the character of the sewage and modify its chemical composition to such an extent that different treatment is required for sewage containing different inorganic impurities. For instance, carbonates of lime cause the property in water known as temporary hardness. Nitrogenous matter in advanced stages of decomposition is present as nitric acid. To form nitrates, and thus complete the process of purification, the nitric acid must have a suitable alkaline base to react upon. Lime in the form of carbonates is a suitable base that readily combines with nitric acid and forms nitrate of lime.

It follows, therefore, that sewage which is moderately alkaline in reaction is the most suitable for purification, while sewage composed of a soft water, like many water supplies from surface sources, must be made moderately temporarily hard before nitrification can take place. In practice, soft waters are made sufficiently hard for nitrification by adding a suitable amount of lime to the sewage. Other bases can be used for this purpose, but lime is generally selected for economic reasons and because it is readily obtainable in any market.

Water containing about 10 degrees of hardness, according to the Clark-Wanklin scale, which is equal to 14 parts or grains of carbonate of lime to 100,000 parts of water, would probably be the best suited for ordinary sewage. Where the sewage receives the acid waste from factories, however, the degree of hardness required to neutralize the acid so as not to interfere with the ordinary process of decomposition would depend upon the amount of acid in the sewage. The permissible degree of hardness in a water supply is 20 degrees Clark-Wanklin, or 28 parts per 100,000, and if this amount is not sufficient to neutralize the acid in the sewage, the deficiency must be supplied by adding lime, soda or some other base to the sewage. Care should be exercised, however, not to add too much lime, as an excess of what is required to neutralize the acid will precipitate the suspended solids and part of the matter in solution, besides temporarily inhibiting the growth and activity of the putrefactive bacteria.

Sewage containing inorganic matter in the form of sulphate of lime, when undergoing decomposition, is liable to produce sulphuretted hydrogen from the reduction of sulphates, and the effluent from purification works containing sulphate of lime, particularly when iron is present in solution in the water, is liable to be of a dark color, while the lime and iron together in the tanks is liable to act as a precipitate and throw down part of the suspended matter in the sewage.

Salt in the form of chlorine is an inorganic constituent of all sewage. It is present in varying quantities in different sewage, but does not seem to exert an unfavorable influence on its decomposition. In fact, there is reason to believe that a certain quantity of chlorine is necessary for the exercise of bacterial activity.

Suspended Matter—Particles of matter in sewage that are visible without the aid of a microscope or magnifying glass, and that are floating in the liquid either on the surface or submerged, are known as suspended matter. All matter in suspension, except the fine particles of clay that impart turbidity to waters, are of organic origin. Cloth, paper, orange peels, etc., are familiar examples of matter carried in suspension by sewage. Coarse particles of suspended matter are found principally in fresh sewage. Before reaching a sewer outfall or purification works, most of the suspended matter in sewage is in a finely disseminated state, but still visible to the naked eye and still in suspension. The disintegration of the coarse particles of matter is brought about by mechanical means, such as scraping along the walls of the sewers, and by a biological process due to micro-organisms.

Colloidal Matter—That portion of the suspended matter of sewage or sewage effluents that is in emulsion is known as colloidal matter. Such matter is of organic origin, belongs to the albuminoids and is invisible. It marks one of the stages in the decomposition of some albuminoids, and its presence in the effluent from sewage purification works indicates that the reduction of albuminoids is incomplete.

Effluents that contain colloidal matter are liable under certain conditions to give rise to subsequent putrefaction. To avoid this, it is fully as important to reduce the colloidal matter as it is to reduce the suspended matter. The amount of colloidal matter in an effluent can be determined by gathering it on parchment. It is a jelly or glue-like substance that very slowly penetrates parchment, which being pervious to water permits it to pass but holds back the colloidal matter.

Matter in Solution—Most of the matter in solution in sewage is inorganic, and consists of dissolved salts of iron, lime, chlorine and other mineral substances. Chlorine is usually present in the greatest quantity, and the amount remains almost constant throughout the process of purification. About 40 per cent. of the matter in solution in sewage is supposed to be organic. This portion of organic matter, however, may be more properly classed as colloidal, for the greater portion of it is in emulsion not in solution.

About 80 per cent. of the soluble or colloidal organic matter in sewage can be and usually is removed or reduced to harmless compounds by the process of purification. Very little of the soluble inorganic matter, however, is changed, and as this portion of the sewage is non-putrescible and non-injurious to health, its reduction is not necessary. The inorganic matter in solution in sewage is of interest only when from its chemical composition it either aids or retards the natural process of putrefaction and nitrification.

Detritus—That portion of inorganic matter like sand, gravel, pebbles, etc., that is washed into the street sewers during rain storms is known as detritus. Detritus is of little importance in sewage purification, as it is non-putrescible, and the small amount that passes the catch basins in the rain-water inlets settles to the bottom of the septic or settling tank, where it simply adds its volume to the sludge.

Gas—Sewage is an extremely unstable compound that is constantly undergoing changes that reduce the organic matter to simpler forms. The reduction of organic matter in sewage is accompanied by an evolution of gases, some of which remain in solution while the greater portion in free state escape to the atmosphere. The gases from decomposing sewage are usually inflammable, and when collected and conducted to a suitable burner, give off light and heat almost as intense as that from ordinary illuminating gas. In tropical countries, where the bacterial activity is greatest and a corresponding amount of gas evolved, the gas can be and has been collected in a gasometer and used for light, fuel and power about the plant. The gas evolved by decomposing organic matter generally consists of the following constituents:

Carbon dioxide	5.90
Oxygen	.76
Methane (marsh gas)	75.18
Nitrogen	17.40
Hydrogen	.26
Undetermined	.50
Total	100

The quantity of gas given off by sewage depends greatly upon the temperature, about twice the quantity being evolved during the summer months as that given off during cold weather. There are two reasons for this. In the first place, the optimum temperature for bacteria, at which they are most active, is that of summer heat; consequently, the greater activity of the bacteria during warm months would result in the evolution of a greater quantity of gas. In the second place, the capacity of sewage to absorb gas is in direct proportion to its temperature, being least during warm weather; consequently a greater proportion of gas would be held in solution during cold weather and would pass off in the effluent.

As the quantity of gas given off by decomposing organic matter is an index of the rate of putrefaction, the reduction during winter months indicates clearly that in order to obtain the best results, means should be provided at purification works in cold climates to protect the sewage and effluent from the weather.

The quantity of gas that under favorable conditions is given off from decomposing sewage varies with the strength and composition of the sewage, and ranges from 3 to 8 gallons of gas during twenty-four hours for each 100 gallons of sewage. A fair average would probably be 4 gallons or about ½ cubic foot of gas per day for each 100 gallons of sewage.

Physical Characteristics of Sewage—When viewed as it leaves the sewer at an outfall or at a purification works, sewage presents the appearance of slop water with some solid matter in suspension. The appearance of sewage varies considerably with its strength and staleness; thus, strong, fresh sewage would contain more matter in suspension than would a strong, stale sewage, while strong sewage generally has a more milky appearance than a weaker mixture. The *strength* of sewage depends on the proportions of organic matter it contains, and the degree of *staleness* depends on the length of time it has been exposed to putrefactive processes. Both of these characteristics are of importance in sewage purification. The strength of sewage determines the amount that can be successfully treated by filtration in a given time, and the staleness of sewage determines the length of time it should be subjected to septic processes.

The condition of sewage best suited for purification depends upon the method employed. If sewage is to be treated at a precipitation works the sewage should be delivered at the plant in as fresh a state as is possible. For septic treatment, on the other hand, the more stale the sewage the less time it requires for bacterial action at the purification works.

Strength of Sewage—Sewage of average strength contains, in addition to the organic and inorganic matter in the water supply, about one part of organic matter and one part of inorganic matter in each 1,000 parts of sewage. In a well designed and operated purification plant, from 80 to 90 per cent. of the organic matter, and all the inorganic matter in solution, can be removed; this leaves in the tanks to form sludge only that portion of the inorganic matter that is in suspension, and which seldom exceeds .05 part in 1,000 parts of sewage.

Sludge—Sludge is the insoluble portion of sewage that cannot be liquefied, mineralized or converted into gases. The composition of sludge depends considerably upon the composition of the sewage and whether storm water enters the sewers. It consists mostly of mineral matter together with some organic matter and about 90 per cent. of water. Sludge is of interest chiefly for the reason that the residue that cannot be liquefied must be removed from the tanks by mechanical means.

Effluents—An effluent is the liquid portion of sewage from a purification plant after the organic matter has been reduced. In practice, the term *effluent* is applied to the outflow from any portion of a purification plant, such as from a septic tank to filter beds. The term *influent* refers to the treated fluid when flowing into or on one portion of a purification plant from another portion. For instance, an effluent from a septic tank would be an influent to filter beds.

Sewage before undergoing the process of purification is sometimes spoken of as raw, or crude sewage, but generally it is known simply as sewage.

Variation in the Flow of Sewage—Sewage varies from day to day and from hour to hour, not only in volume but also in its composition. The variation in sewage is more marked in manufacturing than in residence towns, while at the same time the sewage of no two places is alike, even though in each place it is of domestic character. It is for this reason that the purification and disposal of sewage from each town requires special study and treatment, and rules cannot be laid down that are applicable to all cases.

The volume of sewage to be cared for during rain storms is much greater than the dry weather flow, and this increase in volume usually continues for some time after a storm, during which time ground water finds its way into the sewers. The increase in volume of sewage due to storm water is generally compensated for by the correspondingly weaker sewage to be treated, which permits a greater quantity to be purified in a given time on a certain area of filter surface.

The dry weather flow of sewage varies greatly both in strength and in volume during the twenty-four hours of a day, being greater in the day time, when the waste from factories is added to the domestic sewage. The strength and volume of sewage fluctuates from hour to hour during the day in residence towns, being greatest during those hours occupied in the preparation of meals, while in manufacturing towns great vats of manufacturing fluids might be discharged into the sewers at any time, thus changing in a few minutes both the quantity and chemical composition of the sewage. During the night, when the flow of sewage is light, it is also correspondingly weak, and is made up, to a large extent, of pure water leaking in at plumbing fixtures and ground water which bears a greater proportion to the night than to the day flow of sewage.

The variation in volume and composition of sewage has a marked influence on the construction and operation of a purification plant, the design of which should not be undertaken until data covering such subjects are at hand. Means can then be devised to regulate the flow and bring about a more uniform mixture of the composition.

Temperature of Sewage—The temperature of sewage is an important consideration in the process of purification. The micro-organisms that liquefy organic matter flourish best in a medium with but slight range of temperature, which does not drop below 50 degrees Fahr. Below that temperature little bacterial activity takes place and consequently liquefaction of solids is at a minimum. In cold weather the temperature of sewage varies with and is generally about 10 degrees warmer than the temperature of the atmosphere. During the summer months, the temperature of sewage averages about 75 degrees Fahr., and seldom rises above 80 degrees Fahr. Summer temperature of sewage is the most favorable for the liquefaction of solids. This is evidenced by the fact that organic matter accumulates in septic tanks during winter weather and is destroyed when the temperature rises in the spring and summer. In designing a sewage purification plant, the climate where it is to be constructed should be taken into consideration, and if the mean temperature for December, January and February is below 40 degrees Fahr., to secure satisfactory liquefaction of the solids, all tanks, reservoirs or wells that contain much sewage should be enclosed from the weather. Filter beds, however, that are suitably underdrained need not be protected from the weather. Most of the sewage rapidly drains out of the sand, leaving the interstices filled with air and what liquid remains is quickly thawed when sewage is again applied to the bed.

DECOMPOSITION OF SEWAGE

Bacteria—In nature, organic matter is broken down into inorganic compounds by protozoa, fungi, yeast, moulds and bacteria. In the artificial environment created by a sewage purification works, however, bacteria play the most important part in the reduction. **Bacteria** are minute organisms belonging to the vegetable kingdom, and closely allied to algæ, fungi, yeasts and moulds. They possess no chlorophyl or green parts to decompose carbon dioxide from the air, so they live more like animals than like plants, and secure their food by wresting their cellulose, as well as the other constituents entering into their composition, from animal or vegetable tissue, either living or dead. Bacteria consist of but one cell, which in different species assumes different shapes. Some known as bacilli are slightly curved rods with a length of two or more diameters. Micro-coccos are little spherical shaped bacteria that perhaps are the most minute of all forms, while the spirillum are shaped like the coil of a corkscrew or the spiral of a spring. Bacteria are very prolific, and make up in the rate of reproduction what they lack in size. With plenty of food and in suitable environment, a single bacterium in twenty-four hours will produce 16,000,000 progeny; in two days 24,000,000,000, and in a week the number expressing their quantity would be made up of fifty-nine figures. As a matter of fact, however, conditions are never suitable for this enormous growth. Each specie of bacteria requires certain substances to feed upon, and when that particular substance is exhausted, or nearly so, the bacteria must strive with the more fortunate species, existing in what to them is a favorable environment; and, in the struggle that ensues, the doctrine of the survival of the fittest holds true and one specie of bacteria ceases to exist that the others may live. Even in mediums where the conditions of food, temperature, and moisture are favorable, other conditions interfere to inhibit or retard the growth of bacteria. This, perhaps, is nature's way of holding in check the growth of micro-organisms, which, left unchecked, might become more of a menace than a blessing to the world. Each bacterium in the exercise of its natural functions, excretes a substance that dissolves the material it requires for food, thus liberating, in assimilable form, the particular portion required by the organism.

The solvents elaborated by bacteria, when attacking dead organic matter, are known as *enzymas;* when producing disturbance in a living host, they are known as *toxins* (poisons). These toxins or enzymas, when they exceed a certain percentage of the medium in which the bacteria exist, act as a poison to the bacteria that elaborate them and inhibit their further growth. They do not, however, interfere with the activity of other species of bacteria which can take up the work at this point and carry it a step further towards its final reduction. A familiar illustration of this phenomenon can be cited in the case of alcoholic fermentation of grape juice. When the alcohol elaborated by the alcoholic ferments (yeast) exceeds 19 per cent., the alcoholic ferments disappear and the grape juice, in which the sugar has become converted into alcohol, has become wine. If sterilized and protected from air, the liquid will remain in this condition indefinitely, even though sufficient food remains to support abundant bacterial life; should the wine be left exposed to the atmosphere, however, a new specie of yeast, the lactic ferment, would take possession and convert the wine into vinegar.

So long as the enzymas or toxine of bacteria can be kept below a certain percentage, the organisms will continue to exercise their function of breaking down organic matter. The simplest and most practical way to keep down the percentage of enzymas is to remove the products as soon as they are formed. In sewage purification, by the septic process, this is accomplished by passing a continuous flow of sewage through the tanks, thus providing an adequate supply of food for the bacteria, and at the same time removing the products of their activity.

Bacteria multiply by fission and reproduce by spores. In the process of multiplication, each bacterium grows to about twice its original size, gradually constricting at the center until both parts have attained their full growth, when they part at the constriction and each half starts on an independent existence.

When thrust into a favorable environment, bacteria immediately begin to vegetate, and so long as food is plentiful in an easily assimilable form, they continue to multiply with the production of but little of the solvent fluids that break down the coarser solids. When, however, the bacteria have multiplied to such an extent that the easily assimilable food is rapidly becoming exhausted, or when for any other reason the medium becomes an unfavorable one, the energy of the organisms is required to secrete and elaborate solvent substances to prepare food for their requirements, and the period of multiplication comes to an end. That is the process which takes place in the septic tank, contact bed, or intermittent filter, when sewage is first applied, or when the tank or bed is put in service after a long period of rest. The sewage is seeded with bacteria from the air, water and soil, and a period of vegetation ensues during which the organisms best suited for the purpose multiply to such an extent that the readily obtainable supply of food becomes exhausted. During this period very little reduction of the organic matter takes place, and the effluent is but slightly purified. Following the vegetative stage comes a period of bacterial activity in the bed or tank which is then said to be *ripe*, and the reduction of organic matter and consequent purification of the sewage is at its maximum.

When a medium becomes so unfavorable for a particular bacterium that it can no longer survive, its entire vital energy is directed towards forming a little spore, or seed, which possesses remarkable vitality and can withstand extremes of heat, cold, drought or moisture that would quickly kill the mother cell. This is the little organism's way of perpetuating its species, or of reproducing itself. The spore, containing dormant vitality, remains unaffected by the surviving organisms and continues in that resting state until the proper conditions of heat, moisture and food cause it to germinate and set up a living ferment or putrefaction.

Classification of Bacteria—Bacteria are classified according to their mode of living, as aërobies, anaërobies and as facultatives. *Aërobic* bacteria live only in the presence of free air or oxygen; *anaërobic* bacteria thrive only in the absence of air or oxygen, while *facultative* bacteria accustom themselves either to the presence or absence of free air or oxygen, and live indifferently either as aërobies or as anaërobies. When the more favored habitat of facultative bacteria is in the presence of air or oxygen, they are known as *facultative aërobies.* When they thrive better in the absence of free air or oxygen they are known as *facultative anaërobies.* Aërobic, anaërobic and facultative bacteria each play an important part in the purification of sewage. The obligate anaërobies, that is, those that live only in the absence of free air or oxygen, though they are comparatively few, are exceedingly active and bring about the reduction of solid matter at the bottom of septic tanks and contact beds.

Facultative anaërobies, on the other hand, are less active than the obligate anaërobies, but they are far more numerous, as would be expected, being better suited to live in a medium that contains little or no oxygen, but may at any time become aerated. Obligate aërobies are present in the first and last stages of sewage purification. In the first stage, they prepare the way for the more active anaërobies by exhausting the supply of oxygen in the sewage. In the last stage they play the final part of reducing the nitrites to harmless nitrates.

It should not be inferred that because some bacteria live only in the absence of oxygen, that oxygen is not necessary to their existence. They obtain their oxygen like fish and aquatic animals, by wresting it from the medium in which they exist, or from organic matter contained in that medium, and are destroyed by being thrust into an atmosphere of air or oxygen. For instance, Pasteur found that yeast, which he classed as both aërobic and anaërobic, when introduced to a sugar solution, containing no oxygen, obtained the oxygen required for their existence from the sugar or from the water present.

Bacteria are still further classified according to the food they require, as saprophytes and as parasites. Saprophytic bacteria live on dead organic matter, either animal or vegetable, or on the secretions or excretions of plant or animal life. It is the saprophytic bacteria that are chiefly interesting in sewage purification, as it is due to their activity that organic matter becomes mineralized. Parasitic bacteria can live and thrive only within a living organism. They can exist for a limited time outside of a living host, but their vitality constantly diminishes in the unfavorable environment, until at last they cease to exist.

It is quite conceivable that a third class of bacteria might exist that can adapt themselves to circumstances and live equally well as parasites or as saprophytes, and that under some conditions such bacteria might become pathogenic. Parasitic bacteria that produce functional disturbance in an organism, once they gain lodgment therein, are known as *pathogenic.* All parasitic bacteria, however, are not pathogenic; many non-pathogenic species are present during life in the warm moist lining of the intestines, which, containing a plentiful supply of food, provides a favorable environment for bacterial life.

Parasitic bacteria are only indirectly interesting in the study of sewage purification. No special provision is made at purification works to destroy the parasitic or pathogenic bacteria, the general mineralization of the organic matter being depended upon to destroy the parasites.

Saprophytic or septic bacteria may be distinguished as *putrefactive bacteria,* that is, those which break down nitrogenous substances into ammonic compounds, carbon dioxide and water, and *fermentative bacteria* that break down carbohydrates and other non-nitrogenous substances into carbon dioxide, water and methane gas.

Products of Bacteria—Under favorable conditions, each species of bacteria selects for its nourishment certain suitable substances found in the medium where it exists, and in the process of assimilation excretes a substance peculiar to that particular species, and by which it can be identified. Certain species of bacteria, however, not only are able to avail themselves of different kinds of foodstuffs but the enzymas they excrete differ with the food. For instance, the bacilli of cholera when growing on sugar, which is a carbohydrate, produce butyric acid, the substance that gives butter its rancid taste; but when growing on nitrogenous material, excretes a toxalbumen that is poisonous.

Fermentation of Carbohydrates—To bring about a fermentation of carbohydrates, some nitrogenous material, also some extratives and salts such as are found in the ashes of burnt yeast, must be present in the medium. This material is probably what the micro-organisms live on while producing enzymas that convert starch into sugar. In the alcoholic fermentation of carbohydrates, such as grape juice, the starch must be converted into sugar before a true fermentation or splitting up and hydration can begin. The products of carbohydratic fermentation are alcohol, carbon dioxide, water, succinic acid and glycerine. True carbohydrates are among the most easily reduced substances found in sewage, and less residue is left after the reduction than from any other constituent. The alcohol and carbon dioxide being volatile, escape as gases, while the succinic acid and glycerine, which are present only in very small quantities, are reduced to more simple compounds in the subsequent process of purification through which they pass.

The fermentation of carbohydrates is an anaërobic process carried on in the absence of air; consequently the reduction reaches the maximum in either septic tanks or contact beds, where the supply of oxygen is exhausted. The conditions necessary for the fermentation of carbohydrates are nitrogenous material in a soluble condition, phosphoric acid, and carbohydrate like grape sugar, capable of fermentation.

Cellulose Fermentation—Next to the putrefaction of nitrogenous matter, the fermentation of cellulose is the most important reduction in sewage purification. Rags, paper, vegetable fiber, and other like material, that under ordinary conditions are stable and resist the reducing influences of bacteria for a long time, when placed in a septic tank are quickly broken down and destroyed. For instance, in some experiments conducted by the Massachusetts State Board of Health, at Lawrence, to determine the length of time required to break down cellulose material, a considerable quantity of newspaper, cotton and woolen cloth, contained in a wire basket, was placed in a septic tank and allowed to remain there for one month and twenty-seven days, from October 4 to December 31, 1900. When taken out the cloth and paper were still intact, but so rotten that they fell to pieces when touched.

Cellulose fermentation is an anaërobic process that in nature is going on at all times in manure heaps and in vegetation at the bottom of lakes and ditches, where it is known as methane or marsh gas fermentation. Cellulose fermentation differs from alcoholic fermentation principally in the products of the two ferments. In cellulose fermentation, methane is the chief product, while in alcoholic fermentation alcohol is the chief product.

Before cellulose can be fermented, the organism which effects the change must first invert the cellulose, that is, break it down by means of its enzymas into starch, dextrin and sugar. It is then changed by the true fermentation into methane and carbon dioxide, fatty acids arising as a bi-product.

Decomposition of Albumen—The reduction of nitrogenous or albuminous materials is a progressive process that is both aërobic and anaërobic, and results in the liquefaction of the nitrogenous solids contained in sewage. This process, if carried too far, becomes extremely disagreeable, and gives rise to very offensive odors. Anaërobic decomposition when carried to the disagreeable stage, is known as putrefaction, familiar examples of which may be cited in the case of rotting of eggs and the rank putrefaction that takes place in water-tight cesspools. The products of nitrogenous decomposition are ammonia and carbon dioxide, which may remain in solution or in gaseous form escape to the atmosphere; water, which passes off in the effluent, and nitrous acid, the only organic residue left that required further reduction. Sewage at this stage of decomposition contains no oxygen, and the further purification known as a process of nitrification is effected by aërobic bacteria and must be carried on in the presence of air or oxygen. The conditions favorable to the decomposition of nitrogenous matter are, a temperature above 50 degrees Fahr. and not over 100 degrees Fahr.; moisture; a medium that is not either strongly acid or excessively alkaline, but is preferably slightly alkaline, and conditions that promote the exclusion of air. To bring about this last condition it is not necessary to enclose a decomposition or septic tank, as experience has demonstrated that a thick scum capable of excluding air will form on the surface of open sewage tanks.

Nitrification—The process of nitrification is effected in two stages: first, the formation of nitrites in the form of nitric acid from nitrous acid, and second, the formation of nitrates from the combination of nitric acid, oxygen and an alkaline base. There are certain conditions necessary for the successful nitrification of organic matter. The first requisite is a suitable food. Nitrifying organisms cannot thrive in the presence of large quantities of organic matter, but they can apparently feed upon organic matter, or with equal ease thrive upon inorganic matter. There must, however, in either case be present a certain amount of phosphate. A second requisite is a plentiful supply of oxygen, without which nitrification cannot take place. In fact, if there is a deficiency of oxygen and if sufficient organic matter is present to support the life of dentrifying organisms, a reverse process will occur which instead of building up breaks down with an evolution of nitrogen gas. A further condition for nitrification is the presence of a base with which the nitric acid can combine to form nitrates. Nitrification can take place only in a feebly alkaline medium; an access of alkali will retard the process. The final requirement of nitrifying organisms is a favorable temperature. Nitrifying bacteria can act at as low a temperature as 37 degrees Fahr., but at a higher temperature they become more active. The temperature at which their activity becomes sufficient for ordinary nitrification is 54 degrees Fahr.; their activity then increases with the temperature until 99 degrees Fahr. is reached, after which there is a rapid falling off of the activity until at 122 degrees Fahr. it almost ceases, and at 131 degrees Fahr. it ceases altogether. Strong light as well as high or low temperature is injurious to the process. Darkness or diffused light is good for nitrification. The seasons of the year have some effect on the process of nitrification, which is more active during the growing months of May and June than in the warmer months of July and August.

Denitrification—The reverse process of nitrification will take place in a filter bed or contact bed, if sewage high in ammonia compounds but devoid of oxygen is applied to a bed that is not well aerated. In this reverse process, anaërobic bacteria take the oxygen they require from the nitrates, thus breaking them down into simple compounds with an evolution of nitrogen gas. The oxygen liberated by the organisms combines with the carbon to form carbon dioxide, a portion of which may be evolved as gas, while the remainder combines with a base to form an acid carbonate.

ANALYSIS OF SEWAGE

INTERPRETATION OF SEWAGE ANALYSIS

CHEMICAL ANALYSIS

Object of Sewage Analysis—There are two conditions which are determined by a chemical analysis of sewage: One is the amount of organic and inorganic matter in the sewage, and the other is the stages of putrefaction or reduction the organic matter has undergone. Both conditions, that is, the staleness of the sewage and the amount of organic and inorganic matter contained, are determined by the one set of chemical examinations, which consist of reactions to determine the amount of free ammonia and albuminoid ammonia both in suspension and in solution, the amount of chlorine, the total solids, nitrates, and the oxygen consumed.

In a chemical determination of the organic matter in sewage, the quantities reported are only an index of the total amount and cannot be considered as the total quantity present. For instance, organic matter is made up of various compounds, of which water is the chief constituent, usually averaging as high as 80 per cent. If then 100,000 grains of extremely strong sewage, of which one-half bulk is organic matter, were examined chemically, the organic matter would be determined by evaporating to dryness a quantity of the sewage and weighing the dry residue; as all moisture would have been evaporated from the organic matter, the residue would consist of but 20 per cent. of the original bulk, and this 20 per cent. is all that would be reported by the chemists.

Determination of Odor, Color and Turbidity—In sewage analysis, the odor, color, turbidity and other physical characteristics should be noted. It is evident that an effluent which emits an odor cannot be well purified. Color should not be noticeable in effluents except where dyestuffs from industrial wastes are treated or where the water supply is colored, as in the case of some surface water supplies that are obtained from swampy regions. Turbidity is an indication that the effluent contains large quantities of finely disseminated matter in suspension and that further treatment is necessary for its purification.

Acidity or Alkalinity—Moderate alkalinity is favorable for bacterial purification of sewage, and a determination of the alkalinity of sewage should be made at each analysis, as the acidity or alkalinity of the sewage will in many cases explain the cause of good or poor effluent. For instance, if nitrification were poor, or the effluent from a septic tank cloudy, and at the same time the sewage were acid, the poor liquefaction in the septic tank and poor nitrification in the filter beds could be accounted for by the acidity of the sewage, and means taken to neutralize the acidity by the addition of a suitable alkaline base. If, on the other hand, poor reduction and nitrification are obtained with a moderately alkaline sewage, the cause of the poor effluent must be looked for elsewhere, and the cause remedied.

Temperature—The temperature of both the sewage and the effluent should be taken at the time of collecting samples for analysis. Temperature of the air and condition of wind or calm sunshine or cloudiness are also valuable data under some conditions. As certain conditions of temperature are more favorable than others for bacterial putrefaction of sewage, it is obvious that poor reduction or nitrification can be accounted for by the temperature, which otherwise could not be explained.

Determination of Solid Matter—The total amount of solid matter, both organic and inorganic, in the sewage to or effluent from purification works is determined by the evaporating to dryness of a certain quantity of liquid, and weighing the residue. The amounts of organic and inorganic matter are then determined by igniting the dry residue and subsequently weighing the ash. The amount of ash gives the total amount of inorganic matter in a quantity of sewage analyzed and the difference between the dry residue and the ash, which difference is often reported as "loss on ignition," is an index of the total amount of organic matter present.

The determination of total solids is more important than might at first appear, for it indicates not only the nitrogenous materials both of animal and of vegetable origin, but it also indicates the amount of carbon or non-nitrogenous matter in the sewage, while all other determinations indicate the presence of only one constituent of sewage.

The amounts of solid matter, both organic and inorganic, in suspension and in solution, are determined by passing through filter paper a certain quantity of sewage. The fluid that passes through the filter paper is then carefully evaporated to dryness and the proportions of the dry residue that are organic and inorganic are determined in the same manner as in the case of total dry solids. These indicate respectively the quantity of matter both organic and inorganic that is in solution in the sewage.

The portion of sewage held back by the filter paper is treated in a similar manner, and the amounts determined indicate the quantities of organic and inorganic matter in suspension in the sewage.

The ratio of suspended to soluble matter is an index of the staleness of the sewage, for the fresher the sewage, the more matter in suspension compared with that in solution; and, conversely, the more matter in solution compared with that in suspension, the staler must be the sewage.

Chlorine—Chlorine is present in sewage in the form of chlorides, chiefly of sodium, with less quantities of potassium and ammonium. Chlorine is a constituent of common salt, and besides being used extensively in the household, it enters largely into the composition of all animal tissue, and forms a large percentage of all animal excretions. For instance, as may be seen in Table I, the salts in urine constitute more than one-third of the entire fluid.

TABLE I—CONTENTS OF URINE

Urea	512.4 grains
Extractives (pigment, mucus, uric acid) .	169.5 grains
Salts (chiefly chlorides of sodium and potassium)	425.0 grains
Total	1106.9 grains

The presence of chlorine in sewage, above the normal amount present in the water supply of that locality, indicates animal pollution, and the amount of chlorine above the normal is an index of the extent of pollution. Ordinary water supplies contain but little chlorine, the amount generally ranging from 1 to 2 parts per 100,000. Weak domestic sewage contains about 7 parts per 100,000, strong sewage contains anywhere from 7 parts to 50 parts per 100,000, while sewage of average strength will probably show about 10 parts of chlorine per 100,000 parts of water.

The amount of chlorine in sewage is not affected by purification, and the determination of chlorine in an effluent should correspond approximately with the amount of chlorine in the raw sewage. Any discrepancy in the two determinations, not due to error, that shows less chlorine in the effluent than is present in the sewage, indicates an infiltration of ground water, or dilution from some other source. Frankland* recommends for finding the quantity of ground water diluting sewage, the formula:

$$x=\frac{a-c}{c-b}$$

In which x=the volume of sub-soil or ground water diluting a volume of sewage
a=chlorine in 100,000 parts of sewage
b=chlorine in 100,000 parts of water
c=chlorine in 100,000 parts of effluent

EXAMPLE—With how many volumes of ground water has sewage been diluted when analysis shows 20 parts per 100,000 of chlorine in the sewage, 10 parts per 100,000 in the effluent and the ground water contains 2 parts per 100,000?

SOLUTION—Substituting in the formula the values given in the example:

$$x=\frac{20-10}{10-2}=1.25 \text{ volume of ground water.}$$

* Experimental researches.

Determination of Organic Nitrogen—The organic nitrogen in sewage is determined by testing for free ammonia, albuminoid ammonia, nitrites and nitrates. In these determinate only the total amount of free ammonia is ascertained, while the total amount of albuminoid ammonia, also the amounts in suspension and in solution, are separately determined. The sum of the amounts of albuminoid matter in suspension and in solution should equal the total amount of albuminoid ammonia in the sewage, while the sum of the free ammonia, albuminoid ammonia, nitrates and nitrites indicates the amount of organic nitrogen present.

Albuminoid Ammonia—In fresh sewage, most of the organic nitrogen is present as albuminoid ammonia, and the greater part of it is in suspension. This indicates that but little time has elapsed since the sewage was formed and that but little bacterial action has taken place. In the progressive stages of sewage purification, the next step transforms the albuminoid ammonia from suspension into solution. Albuminoid ammonia in solution when in greater quantity than in suspension then indicates bacterial activity in the right direction.

Free Ammonia—From albuminoid ammonia in solution the nitrogenous matter in sewage becomes and is determined as free ammonia both in suspension and in solution. The albuminoid ammonia in sewage decreases until the organic nitrogen is practically converted into free ammonia. This condition marks the final stage of anaërobic purification. Ammonia contains no oxygen, while nitrates contain a considerable amount of oxygen; therefore to carry the process of purification or mineralization further requires a well aerated environment; and an effluent in which the nitrogenous matter has been reduced to free ammonia indicates that further septic or anaërobic action would be detrimental, causing it to putrefy instead of purify.

In Table II are shown average analyses of effluents from a septic tank at the Experimental Station of the State

TABLE II—FREE AND ALBUMINOID AMMONIA IN SEWAGE EFFLUENTS

Parts per 100,000								
Year	Temperature	Ammonia				Chlorine	Oxygen	Bacteria per Cubic Centimeter
		Free	Albuminoid					
			Total	In Solution	In Suspension			
1898	57	4.86	0.41	0.32	0.09	10.11	2.29	324,500
1899	57	4.03	0.34	0.25	0.09	7.00	2.52	577,100
1900	56	4.61	0.39	0.25	0.14	9.93	2.85	1,209,500
1901	56	4.90	0.43	0.29	0.14	10.40	3.12	671,000

Board of Health at Lawrence, Mass. This table is interesting chiefly in that it shows the almost entire reduction of albuminoid ammonia in suspension, the comparatively small amount in solution and the large amounts of free ammonia present. The data in the table indicate a good effluent ready for aeration.

Nitrites—The formation of nitrites marks the first stage in the oxidation of nitrogenous materials that have become converted into ammonia compounds by the process of decomposition. Nitrites are found only in traces in fresh sewage, and increase in quantity as the sewage becomes stale. An increase in the quantity of nitrites is always accomplished by a corresponding decrease in the nitrogenous compounds, free and albuminoid ammonia. Nitrites are very unstable and in the presence of air, phosphates, moisture and an alkaline base are quickly converted into more stable nitrates.

Nitrates—The formation of nitrates marks the final stage in the purification of the organic nitrogen contained in sewage, and the determination of nitrates is of great importance in sewage analysis. Fresh sewage, as a rule, contains no nitrates and only traces of nitrites, while purified effluents from filter beds should be high in nitrates. As the process of oxidation progresses, that is, at different depths in a filter bed, the nitrates should increase and the ammonia compounds correspondingly decrease.

Oxygen Consumed—The determination of oxygen consumed is for the purpose of ascertaining the amount of carbonaceous or non-nitrogenous matter in sewage. All of the determinations before mentioned are for the purpose of finding the amount of organic nitrogen present, but gave no indication of the presence of hydro-carbons or other non-nitrogenous compounds. The greater the amount of carbonaceous matter in sewage, the greater will be the quantity of oxygen consumed. For instance, in very pure spring or lake water no oxygen will be consumed. In Lake Superior water from .1 to .2 parts of oxygen to 100,000 parts of water might be consumed, while sewage that contains large quantities of carbonaceous matter might absorb as high as 150 parts of oxygen to 100,000 parts of water.

BIOLOGICAL EXAMINATIONS

Bacteria in Sewage—The degree of reduction that sewage has undergone can approximately be determined from the number of bacteria present. In fresh sewage they are most numerous, in some cases numbering as high as forty-five million per cubic centimeter. As sewage passes through successive stages of purification, the food supply of various species of bacteria becomes exhausted and they cease to exist; thus the number of micro-organisms is constantly growing less until in the effluent from septic tanks they will average about 600,000 per cubic centimeter, and in the effluent from a filter bed they will average about 100,000 per cubic centimeter, although under favorable conditions they might become reduced to 2,000 per cubic centimeter.

Pathogenic bacteria in sewage are exposed to more unfavorable conditions and to more unsuitable environment than the saprophytic bacteria, and it is presumed that any process of purification which reduces the number of saprophytic bacteria will reduce in an even greater ratio the parasitic bacteria. As the pathogenic bacteria in sewage are but a small per cent. of the total number present, the number of saprophytic bacteria found by a biological examination of sewage is a fair index of the number of pathogenic variety that might be present.

TESTS FOR EFFLUENTS

Incubation Test—Tests are now generally made of the effluent from purification works. The one commonly applied is known as the incubation test, and is founded on the principle that a good water, free from organic matter, contains oxygen and will not stagnate or become foul if kept out of contact with air. Pure water being saturated with air will not absorb oxygen, but impure water, sewage or sewage effluent that contains organic matter, if corked in a bottle and placed in an incubator, which for five days is kept at a temperature of 80 degrees Fahr., will upon being exposed to the permanganate test absorb an amount of oxygen proportionate to its impurities.

An effluent capable of passing the incubator test is required only when it is to be discharged into a dry ditch or open channel leading to a water course. When an effluent is discharged directly into a stream that has a dry weather flow of seven times the volume of sewage, the purification of the effluent need not be carried to the extent of passing the incubation test, as further purification will take place in the stream.

Fish Test—Fish cannot live in water that is saturated with chemicals, contains poison, or is devoid of oxygen. An illustration of this may be cited in the case of the Passaic River in New Jersey, where fish have been killed or driven away by the rank pollution of the river by sewage. It is a safe provision, therefore, to require the effluent from purification works to be of such a character that fish can healthfully live in the undiluted effluent. Then when effluent is mixed with the water of a river and diluted with several times its volume of water, it will be perfectly harmless to fish and not likely to cause other nuisance.

METHODS OF SEWAGE PURIFICATION

SEPTIC PURIFICATION OF SEWAGE

PRINCIPLES OF SEPTIC PURIFICATION

THE SEPTIC TANK

Bacterial and Physical Action in Septic Tanks—In the septic process of purification, sewage is passed slowly through a vessel or reservoir, called a septic tank, which contains a large body of sewage undergoing the various processes of fermentation and putrefaction. The tank is built in such manner as to promote the growth and activity of the saprophytic bacteria on which the process depends for the reduction of the organic matter in the sewage. When discharged into a septic tank, sewage undergoes a physical separation or sedimentation, in which the heavier particles are drawn to the bottom by the force of gravity and there contribute to the accumulated sludge, while the lighter particles float to the top of the liquid, thus forming a scum on the surface and leaving the intermediate depth comparatively clear. At this stage of purification the sewage contains a greater or less amount of air or free oxygen, according to the length of time it has been in the sewers and subject to bacterial action. The aërobic bacteria in the sewage, however, rapidly deprives the liquid of its oxygen, while at the same time the layer of scum on the surface, and the aërobic bacteria therein contained, prevent oxygen from the atmosphere from penetrating to the lower depth of the tank. The sewage thus being deprived of oxygen, is in suitable condition for anaërobic action, which is the most effective in liquefying solids, and the tank then becomes the seat of two very distinct actions. In the interior and on the bottom of the tank anaërobic bacteria attack the solid matter, both nitrogenous and carbonaceous, and convert it into simple compounds suitable for the requirements of aërobic bacteria. On the surface of the liquid, in the presence of air, on the other hand, aërobic bacteria are busy reducing to still simpler forms the products liberated by the anaërobic bacteria, and at the same time aërobically reducing the scum on the surface of the sewage.

The sludge on the bottom of the tank is subjected to a physical as well as to a biological action. Gases produced by the liquefying bacteria in the sludge, in rising to the surface of the liquid, entangle or saturate, as the case may be, some of the solid matter on the bottom. The solids so affected, buoyed up by the gases, rise slowly toward the surface of the sewage. When near the surface the gas becomes liberated and the solids again are carried by the force of gravity to the sludge in the bottom of the tank. This physical action is going on constantly, night and day, in a septic tank. Bubbles of gas carrying sludge can at any time be seen rising to view in the tank.

During the vertical movement of sludge in a tank it is carried in a horizontal direction toward the outlet, a distance proportioned to the rate of flow through the tank, and the time consumed in traversing the vertical distance from the bottom of the tank to the surface of the fluid and back again to the bottom of the tank. For example, in a tank 6 feet deep to the surface of the liquid and 96 feet long, through which sewage flows at the rate of 4 feet an hour, if solid matter consumed five minutes in its vertical movement from the bottom of the tank to the surface of the liquid and back again, it would be carried during each vertical course one inch toward the effluent outlet, and as the distance traversed in each oscillation would equal $6 \times 2 = 12$ feet, the entire distance traversed before reaching the outlet would be $12 \times 1152 + 96 = 13{,}920$ feet, or almost 2½ miles.

The septic-tank process of sewage purification is not a complete process in itself, but only a preparatory step that almost completely reduces the carbonaceous matter, but only partially purifies the nitrogenous matter contained in the sewage. The effluent from a septic tank is entirely devoid of oxygen, and when freely exposed to air by falling in a cascade over baffle plates, or sprayed into the air, it almost instantly will absorb 70 to 75 per cent. of the air it can absorb. Subsequent treatment of septic effluent therefore should consist, first, of a thorough aeration and then an intermittent application in thin films to porous, well-aerated soil, sprayed continuously over filter beds of coarse material, or subjected to aërobic treatment in contact beds, so nitrifying bacteria can reduce the ammonia compounds to stable nitrates.

The odors in the neighborhood of septic tanks are usually noticeable at some little distance from the tanks in the direction in which the wind is blowing. Ordinarily, however, the odors are not seriously objectionable even on the site of the purification works. Bad odors generally result from the treatment of sewage containing relatively high quantities of sulphur compounds, such as the acid waste from factories. They may likewise arise from the treatment of sewage from communities in which the water is hard, due to sulphates of lime or magnesia.

Covered and Uncovered Septic Tanks—So far as the effectiveness of septic tanks is concerned, there is no difference between the results obtained in mild climates by the use of closed tanks and those obtained by the use of open tanks. In open tanks, as in closed tanks, the scum that forms on the surface of the liquid, together with the aërobic bacteria existing within the scum, prevents oxygen penetrating to the lower depth of the tanks. The scum, furthermore, forms a screen to shut out light which would interfere with the bacterial processes within; the turbidity of the liquid also prevents light rays from penetrating to any great depth, even though they should pierce the upper crust or find entrance where the crust is broken or removed. It is probable that on the liquid in open septic tanks a thicker crust will form than is likely in a closed tank, where darkness encourages aërobic action on the surface scum that no doubt is interfered with in open tanks by the direct rays of the sun. A thick scum on the liquid in open septic tanks no doubt is beneficial, not only in excluding air and light, but also in maintaining an even temperature, throughout the year, of the sewage contained in the tank. As a rule, a thicker scum will form during the summer months than during winter weather.

A comparison of the results obtained by treating in open and in closed septic tanks sewage taken at the same time from a common source, can be found in Table III, which shows the average results obtained by Mr. Fowler, of Manchester, England, in a series of analyses made daily for a period of one month. The results, as may be seen, are almost identical.

While there is no difference in efficiency between open and closed septic tanks operated in mild climates, in cold

TABLE III—COMPARISON OF RESULTS FROM OPEN AND CLOSED SEPTIC TANKS

Parts in 100,000	Open Tank	Closed Tank
Free ammonia	3.20	3.10
Albuminoid ammonia	0.50	0.51
Oxygen consumed	8.46	8.43
Chlorine	16.40	16.10

climates the closed tank will be found to possess distinct advantages over open tanks. The advantages consist in the exclusion of snow, which in northern latitudes is no inconsiderable amount, and in the maintaining of a higher temperature more suitable for the activity of bacteria than can be maintained in an open tank. In mild climates, when the additional expense of covering septic tanks need not be considered, a closed tank will prove the more satisfactory, as it conceals from sight the fermenting and putrefying mass of sewage, prevents the nuisance of odors, protects the sewage from flies, and is a protection against wind and rain. In warm climates a wooden covering freely ventilated will prove quite satisfactory. Covered tanks should be provided liberally with ventilation ducts or outlets for the escape of gas liberated by the decomposition of sewage. If provision is not made for the removal of gas as formed, when a sufficient quantity has accumulated, if accidentally ignited it is liable to cause a disastrous explosion.

CONSTRUCTION OF SEPTIC TANKS

Description of a Septic Tank—A small covered septic tank is shown, diagrammatically, in Fig. 1. The tank is

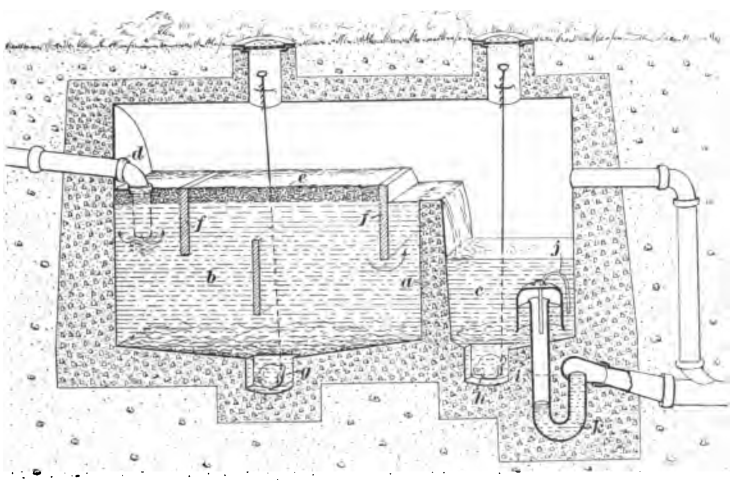


Fig. 1

separated by a wall, *a*, into two compartments, the septic tank proper, *b*, and a collecting and discharge chamber, *c*, generally called a dosing chamber. Sewage enters the septic tank through the inlet, *d*, which is turned down and submerged so sewage will not disturb the surface scum, *e*. From the septic tank, the effluent overflows the weir, or wall into the dosing chamber. One of the baffle boards, *f*, deflects the flow of sewage towards the bottom of the tank, while the other, which extends down about 3 feet below the surface of the liquid, prevents the surface scum from being washed over with the effluent and insures the discharge from the tank being taken from near the center level where the sewage is most clear. When baffle boards are used they should be spaced about 10 feet apart. The board nearest the inlet should project a few inches above the line of flow and to within 2½ feet of the bottom of the tank. The middle baffle board should be set with its upper edge about 18 inches below the surface and its lower edge about 18 inches above the bottom of the tank. The scum board near the outlet should extend a few inches above high water level, and the bottom edge should be midway between the surface of sewage and the bottom of the tank. A valved sludge pipe, *g*, provides means for draining off sludge from the tank without throwing the tank out of service. The floor of the tank is made sloping towards this outlet to facilitate the washing out of the sludge. A sludge pipe from a septic tank is desirable only when the tank is situated at such a level that the sludge can be flushed by gravity to sludge beds located at a lower elevation, or, in case the sludge cannot be flushed by gravity, when the purification plant is of such a size as to warrant the installation of sludge pumps.

The dosing tank, *c*, can be omitted from a septic tank when the effluent is discharged into a stream; when, however, the effluent is subsequently treated by filtration, as effluents invariably should be, the dosing chamber should be so proportioned to the filter beds that one dose will properly flood the filter area. The floor of the dosing chamber is made sloping towards the center where is located the valved outlet, *h*, which should be cross-connected to the discharge to the filter beds, and to a sewer outfall, so that the effluent can be discharged direct at the place of disposal, or supplied continuously to the filter beds during repairs to the automatic siphon. The wall separating the septic compartment and dosing chamber should be made of sufficient strength to hold back the dammed-up liquid in the septic tank, and withstand the varying strains caused by slowly filling and quickly emptying the dosing compartment. The siphon apparatus shown in the illustration is a Miller Automatic Siphon, and is operated in the following manner: When sewage overflows the wall, *a*, into the dosing chamber it rises in the bell of the siphon and overflows into the trap, which it seals, thus confining the air in the space, *i*, which forms the long leg of the siphon. As sewage effluent then rises in the dosing chamber it compresses the confined air in the long leg, thus forcing the water down on one side and up on the other, as shown in the illustration, until the compressed air in *i* is just about to escape under the bend that forms the dip of the trap. Any further flow of liquid into the dosing chamber will then compress the water in *i* so that the confined air can escape from the trap carrying with it some of the water; as the air escapes from *i* the space fills with water from the dosing chamber, thus filling the long leg of the siphon which immediately is thrown into operation and aspirates the contents from the dosing chamber. When the effluent in the dosing chamber is lowered to the level of the mouth of the air pipe, *j*, the siphonage is slowly broken by the admission of air through the pipe. In addition to serving as a vent to break the siphon, the air pipe, *j*, permits the escape of air from the space, *i*, when sewage is flowing in to fill the dip of the trap, a condition that is necessary for the successful operation of the siphon.

When effluent in the dosing chamber reaches the level of the mouth of this air pipe the mouth of the pipe becomes sealed, thus confining the air in the long leg of the siphon. The confined air in the long leg of the siphon then becomes compressed, and depresses the water in the trap in direct proportion to the rise of sewage in the dosing chamber.

The depth of liquid in the dosing chamber that will cause the automatic siphon to discharge, depends on and can be gauged by the depth of water in the short leg, *k*, of the trap.. It is obvious that the column of water inside of the bell will balance that on the outside of the bell, and that the column of water, *k*, in the short leg of the trap will balance an equal column of water in the dosing chamber above the level of water in the bell. Any increase in the depth of water in the dosing chamber destroys the equilibrium of the two columns and brings the siphon into full action. It follows therefore that the short leg of the trap must be of just such length as there will be depth of liquid above the bell in the dosing chamber when the siphon is brought into operation.

The short leg of the trap projects abruptly into the discharge pipe to permit the instantaneous escape of water from the outlet. This is a necessary provision without which water would not escape with sufficient velocity to disturb the equilibrium of the two columns and set the siphon in operation.

An overflow pipe is provided to carry off the effluent in case the siphon becomes obstructed. This overflow pipe serves also as a vent pipe through which air can circulate from the outlets at the filter beds or other place of disposal, up to and through the perforated covers of the manholes.

Capacity of Septic Tanks—Ordinarily, septic tanks should have a capacity equal to the dry weather flow of sewage for twenty-four hours. The dry weather flow is assumed as a basis, because during rainy weather or immediately after a storm, rain water entering through rain-water openings to the street and from rain leaders on buildings, also ground water infiltering to the sewers from the soil, add considerably to the bulk of sewage to be handled at a purification works. If plants were made large enough to care for storm water in addition to the usual domestic sewage, they would be too large for ordinary purposes; so to obtain the best results, provision is made to purify the ordinary dry-weather flow, and additional provision made to care for the storm water without subjecting it to the usual purification process. In providing for the dry-weather flow, it should be borne in mind that tanks sometimes sludge to such an extent that only about 75 per cent. of their capacity is available for storage, and this limitation in size should be taken into consideration when proportioning a tank. The character of the sewage will have a great deal to do with the amount of sludge formed, and where a sewage is strong and contains an unusual quantity of cellulose material, an additional capacity of about 25 per cent. would probably be advisable, while with a weak domestic sewage of ordinary composition, a tank with a gross capacity of twenty-four hours' dry weather flow will no doubt be found sufficient. Many tanks have been installed with capacities of only twelve to eighteen hours' flow, but with few exceptions they have been found too small, and under the most favorable conditions are producing effluent much inferior to those with twenty-four hours' capacity. Too long storage, on the other hand, must be avoided. In some cases where the effluent is to be subsequently treated by sprinkling filters followed by slow sand filtration, a period of sedimentation or septic action of ten or twelve hours might prove sufficient, and permit the treatment of a greater quantity of sewage with the production of a satisfactory effluent.

Velocity of Flow through Septic Tanks—In the septic process of purification, in order to secure the best results, it is necessary that the sewage flow continuously into the tank, to provide a constant supply of food for the reducing micro-organisms, while at the same time the effluent from the tank is continuously withdrawn to carry off the toxic enzymes elaborated by the bacteria. The flow of sewage through the tank, while continuous must not be at too great velocity, or sufficient time will not elapse during the passage for sedimentation and bacterial action to properly perform their functions. It has been found in practice, both in the treatment of turbid waters and in the purification of sewage, that twenty-four hours is a sufficient length of time for sedimentation. Any particles that will settle will do so in that time, while a greater period of time adds considerably to the size of tank required without affecting a corresponding clarification. Furthermore, a longer period of storage in a septic tank is liable to carry the putrefactive process so far that the products will check the reducing organisms and produce an effluent difficult to nitrify. This is more liable to occur when treating a strong than when treating a weak sewage. In practice, the length of time that sewage remains in a septic tank varies from eighteen to twenty-four hours. In some improperly proportioned purification works, a much less period of sedimentation is provided for with correspondingly inferior results. The length of time required for sedimentation and bacterial action in a septic tank varies somewhat with the staleness of the sewage. Ordinarily, for fresh sewage, it may be stated as a rule that twenty-four hours' time should be allowed; on the other hand, when the sewage has flowed through sewers for three or four hours, and arrives at the purification works in a stale condition, a corresponding length of time can be deducted from its length of stay in or passage through the tank. In designing a purification plant, however, it is good provision to allow for a flow of twenty-four hours through the tank, then, upon the growth of the community, when sewage must be passed through at a higher velocity, it can be so done without materially decreasing the effectiveness of the purification.

The actual velocity of flow through a tank should not be greater than one inch per minute, and should be at least one-half inch per minute.

Uniformity of Flow in Septic Tanks—Not less important than the velocity of flow is the uniformity of flow through septic tanks. If arrangements are not made to insure uniformity of flow throughout the whole cross section of the tank, the sewage is likely to flow in a narrow stream or channel with greater velocity than if the flow were uniform. In order to insure uniformity of flow, the inlet to a septic tank should be so designed that the sewage will be spread over the entire cross section. This is best done by providing several branch inlets to the tank, opening at the same level but at various distances from the center line. Such an inlet not only distributes the sewage uniformly, but reduces its velocity on entering so as not to produce eddies or currents to disturb the sludge. The effluent from a septic tank usually flows over a weir which extends clear across the tank, this insuring a uniform flow from the entire cross section.

In Table IV will be found the average purification affected by different lengths of flow through open septic tanks.

TABLE IV—AVERAGE OF ANALYSES ILLUSTRATING THE EFFECT OF DIFFERENT RATES OF FLOW THROUGH OPEN SEPTIC TANKS (LEEDS, 1905)

	12 Hours' Flow		24 Hours' Flow		48 Hours' Flow		72 Hours' Flow	
	Parts per million	Purification (per cent.)	Parts per million	Purification (per cent.)	Parts per million	Purification (per cent.)	Parts per million	Purification (per cent.)
Total solids	1,250	......	1,110	......	1,120	......	1,050	......
Suspended solids	272	52	162	71	155	73	141	76
Nitrogen as—								
Free ammonia	18.2	22	17.5	24	18.8	19	20.8	37
Albuminoid ammonia	6.3	50	5.2	58	4.5	64	4	52
Oxygen consumed in 4 hours at 80° F.	74.2	45	68.8	49	61.2	55	51.1	55

Shape and Dimensions of Septic Tank—There is no certain shape which, better than another, conduces to the efficiency of a septic tank. For economic reasons, however, tanks are built square in preference to rectangular and are built round in preference to oval, as these shapes enclose greater areas for equal lengths of wall. To conform to the topography of the land or to the shape of a purification field, septic tanks are sometimes made irregular in shape or in depth. This irregularity, however, while it adds to the cost of the plant, in no way affects its efficiency.

Within certain limits the depth of a septic tank can be adapted to suit the requirements of each installation, provided it is made not over 10 or 12 feet deep. If a tank is made of greater depth, difficulty will be experienced in securing a uniform movement of the sewage, unless submerged baffle plates are provided to deflect the fluid. Furthermore, a greater depth to a tank will increase the difficulty of removing, and the power required to remove, the sludge, either by hand or by power, or the tanks would have to be located at a higher elevation to secure gravity discharge of sludge to the sludge beds. As the sewerage system grades from the entire drainage area towards the purification plant, which usually is situated at a very low level, near the place of disposal, the depth that a tank can be extended generally is limited by the available fall from the bottom of the tank to the sewer outfall, or to the filter beds.

The construction of tanks that are too shallow should also be avoided, as under such conditions the flow of sewage is liable to stir up the sludge and carry off part of it in the effluent. Five feet is probably the least depth that will produce satisfactory results.

Reduction of Sludge in Septic Tanks—There is no way of predetermining the amount or percentage of sewage that will be reduced or destroyed in a septic tank. It is fair to assume, however, that under the most favorable conditions a tank rightly proportioned with sufficient depth, and having a capacity of 24 hours' storage, through which the sewage flows with a velocity not greater than one inch per minute, if maintained at a uniformly high temperature, will reduce at least 90 per cent. of the sludge in domestic sewage. Under ordinary conditions a septic tank will reduce from 70 per cent. to 90 per cent. of the sludge in an average domestic sewage, free from manufacturing waste. Where conditions are more favorable and the septic tank is properly designed, the sludge will entirely disappear. Such has been the case at Lawrence, Mass., where the sludge is entirely reduced and where for over four years no sludge has been taken from the tanks. It should be remembered, however, that for the first three or four months that a tank is in service there will be an accumulation of sludge during the period of seeding and ripening. This period can be reduced somewhat by seeding a new tank with surface scum and sludge from a ripe tank.

In the average septic tank treating domestic sewage, it can be assumed that of the matter suspended in the sewage, about one-third passes through the tank and appears in the effluent; about one-third is liquefied or gasified, and the remaining one-third remains as sludge. The sludge, however, in most cases can be reduced to about one-tenth by proper design; and in localities where the temperature of the sewage is 78 degrees to 90 degrees Fahr. there will be a complete reduction of the organic matter.

SEPTIC TANK DETAILS

Detritus Tank—In most systems of sewage purification, whether septic tanks, intermittent filters, contact beds or chemical precipitation plants, a tank known as a detritus tank is provided, in which to catch heavy mineral or metallic substances that might have been washed into the sewers and carried along by the force of the flowing liquid to the purification works. Detritus tanks are not intended to store any considerable quantity of sewage, but are more in the nature of traps with a depression below the level of the bottom of the sewer. When anything of greater specific gravity than water reaches the detritus tank, it immediately sinks to the bottom where it remains until removed by mechanical means. A detritus tank should be of sufficient length so that particles of but slightly greater specific gravity than water cannot be carried across the tank by the momentum of the sewage. The outlet from a detritus tank to the purification works generally is provided with a screen to hold back any large floating objects that are difficult to liquefy and that if not removed might interfere with the proper working of the plant.

Dosing Tanks—The size of a dosing tank depends to a great extent upon where the effluent is to be discharged. If the effluent is to be discharged into a stream or lake, the siphon and dosing chamber may be dispensed with and the effluent allowed to flow continuously into the water. If, however, a dosing chamber be used, the siphon should be set to discharge at frequent intervals, as small doses at frequent intervals would conduce to better dilution of the effluent. When, on the other hand, the effluent is to be treated subsequently by filtration on intermittent filter beds, the dosing tank should have a capacity in proportion to the size of the filters. For instance, if the dosing chambers have a capacity of six hours' storage, the filter beds should be proportioned to hold a six-hour discharge.

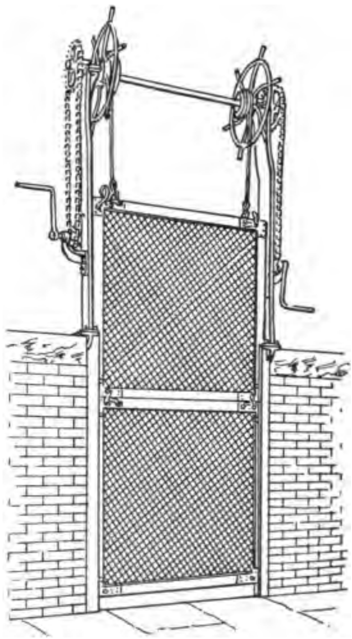


Fig. 2

In calculating the period of storage in septic tanks, the time required to fill the dosing chamber may be deducted from the total period. The dosing chamber shown in connection with the septic tank, Fig. 1, is extended the full depth of the tank. This depth, however, is unnecessary, and, when there is but little fall from the septic tank to the filter beds, the dosing chamber can be made shallower and make up in area the capacity required. When necessary, the bottom of the dosing chamber can be within 12 or 18 inches of the level of the outlet to the tank.

SEWAGE SCREENS

Removable Screens—Sewage screens for small purification plants are generally some modification of the screens shown in Figs. 2, 3 and 4. It is a point of economy when specifying screens to stipulate that they be of some non-corrodible material that will not quickly be eaten away by the chemical action of the sewage. Some purification plants have a lattice work made of iron bars bedded in the

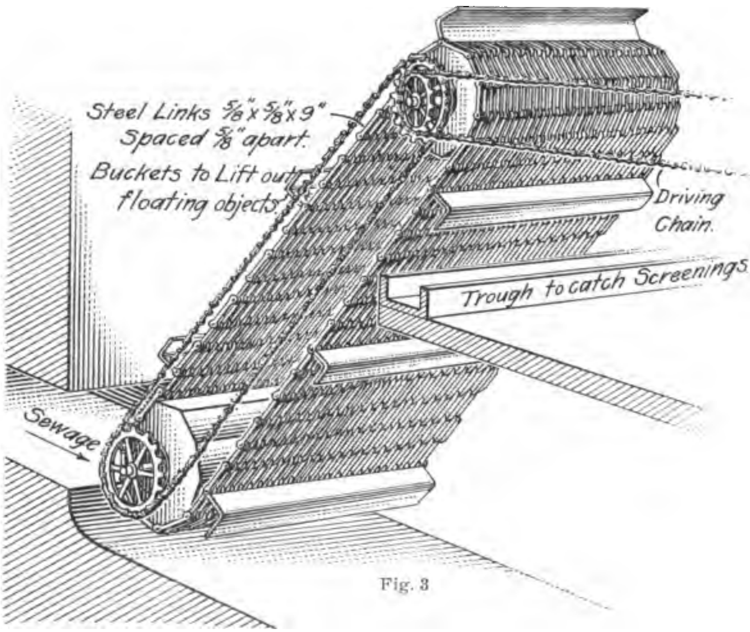


Fig. 3

masonry. With such an arrangement, the screen must be kept free from accumulations by removing the clogging material by hand labor. This is usually done by scraping the accumulations away with hand rakes, a process which generally entails considerable labor. With removable screens on the other hand, when one screen becomes clogged with deposits that screen can be removed, another screen substituted, and the old one laid out to dry, as drying facilitates the removal of the deposits. A removable screen is shown in Fig. 2. Usually two or more sets of grooves are provided, so that a fresh screen can be lowered into place before the old one is removed.

Mechanically Operated Screens—Mechanically operated screens have been successfully operated at large plants where power is required for other purposes. The expense of operating mechanical screens would hardly be warranted however in plants of small capacity or where sewage with but little coarse matter is treated. A mechanical screen that is in use in Glasgow, Scotland, is shown in Fig. 3. This screen is inclined at an angle of 45 degrees, and is run at a speed of 14 revolutions per minute. The face of the screen is provided with buckets to lift out floating objects which are carried over the top axis and deposited in a trough, from which they are pushed by hand into a bucket located in a sump. The screen is operated by chains and sprocket wheels.

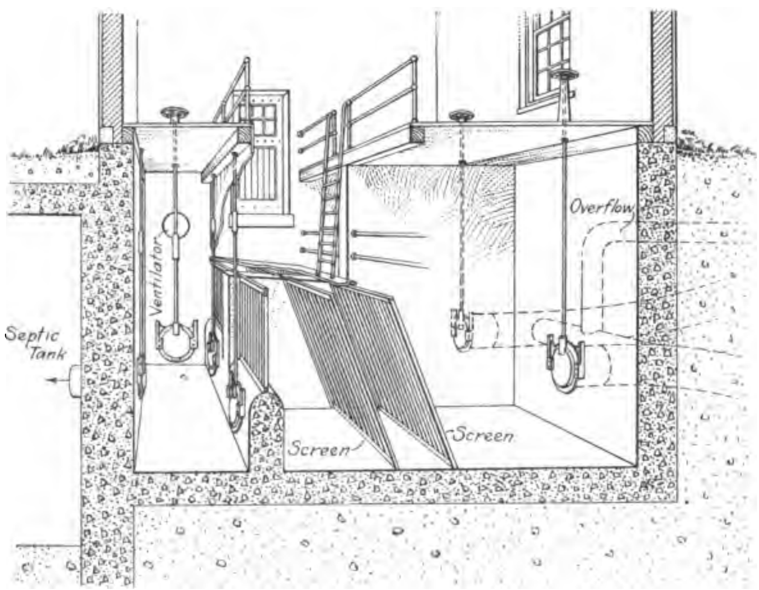


Fig. 4

Detritus Tank and Screen Chamber—An illustration of a detritus tank and screen chamber is shown in perspective in Fig. 4. Sewage enters the tank through the trunk sewers which are cross connected with an overflow pipe that can be used when necessary as a storm overflow.

Each inlet is separately controlled by a sluice valve operated from the floor above the tank, so that sewage can be shut off from either tank and crossed over into the other, thus allowing for cleaning and repairing. Sluice valves likewise control the outlets to the several septic tanks, so that sewage can be discharged into any, all or none of them. The detritus tank is formed by the dwarf wall or weir, which prevents heavy particles from passing through to the collecting gallery and from there to the septic tanks.

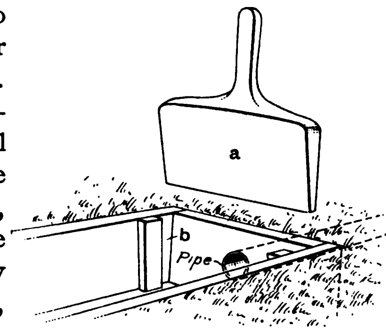


Fig. 5

Sluice valves are provided in the dwarf wall so the detritus tanks can be drained. In operation, however, these valves are kept closed.

Three sets of removable screens are provided in each screen chamber. The number of screens can be varied, however, to suit each installation, as can also the size of mesh in the screens. For extremely large plants more screens and larger mesh screens would be used than for small domestic sewage plants. For ordinary size disposal works treating domestic sewage, screens of about 1-inch mesh will be found satisfactory. The trunk sewers should be bi-passed around the screen chamber and septic tanks so the crude sewage can be discharged into the filter beds or into the place of final disposal without being purified. The bi-passes, none of which are shown in the illustration, would have to be provided with gate valves to control the flow of sewage through the various branches.

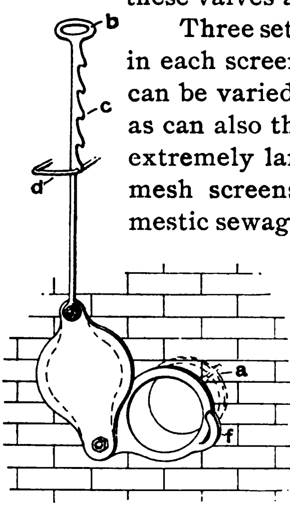


Fig. 6

SLUICE GATES AND VALVES

Shear Gates—The various pipes, ducts and flumes around a purification works are valved so that the flow of sewage can be regulated and controlled, thus making it possible to shut off the flow entirely or regulate the flow to as small a stream as desired. For this purpose various kinds of sluice gates and gate valves have been designed to meet the requirements of the various conditions under which they are used. A simple shear gate used at many small purification plants is shown in Fig. 5. This consists of a wooden blade, *a*, made wedge-shaped, so that when forced down in the groove, *b*, the face of the blade or paddle will press firmly against the face of the pipe and cut off the flow. This form of shear is sometimes used to shut off or regulate the flow of sewage in flumes and distributing troughs to filter beds. The cast-iron shear gate shown in Fig. 6 is an improvement over the wooden paddle or blade, and can be used to control sewage under high pressures. This shear valve can be used in small septic tanks to control the inlet, outflow or the sludge pipe.

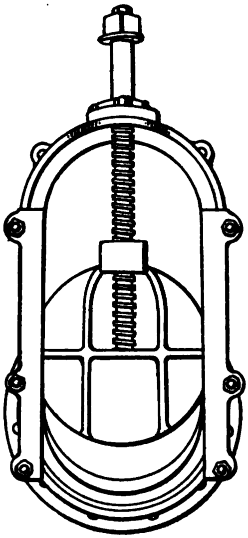


Fig. 7

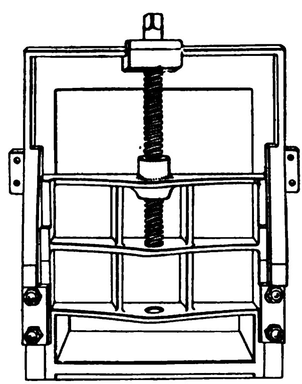


Fig. 8

The sleeve, *a*, can be built into the masonry or bedded in concrete to make a tight joint at this point. The gate can be opened or closed by means of the handle, *b*, which is notched at *c*, to catch on the guard, *d*, and thus lock the gate partly open. The guard prevents the handle falling over to one side in the tank, and the catch, *f*, holds the gate firmly against the face of the sleeve when the valve is closed, thus completely shutting off the flow of sewage.

Sluice Gates—In large size purification plants where

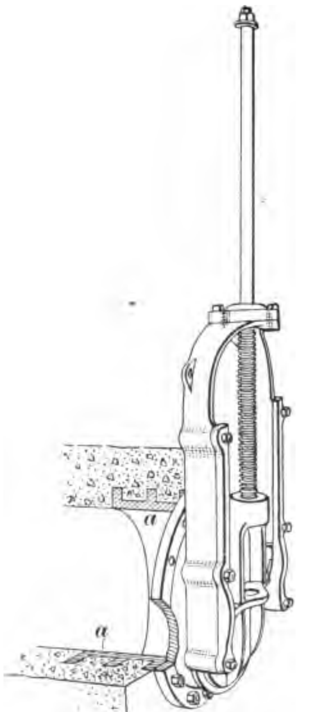


Fig. 9

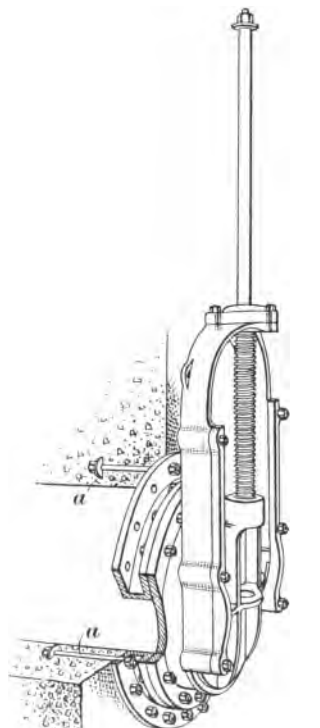


Fig. 10

the sewer main and effluent pipes are large, a stronger and better gate than the shear gate is required to properly control the flow of sewage. For this purpose sluice gates, Fig. 7, are generally used. These gates are bronze mounted, may be had with extension screws and in round, rectangular or square patterns. A square-pattern sluice gate is shown in Fig. 8. These patterns can be had in stock sizes up to 4 feet in diameter for round sluice gates and 4 x 4 feet for square gates.

Sluice Gates with Thimble Set in Concrete—There are three different ways of connecting sluice gates to the walls of a tank or to the piping entering a tank. When a sluice gate is to be set against a face of cement concrete, a thimble, *a*, Fig. 9, is usually bedded in the concrete and the sluice gate bolted to the face of the thimble.

Sluice Gates Anchored to Masonry—Sluice gates are sometimes attached to concrete walls in the same manner that they usually are connected to brick or masonry walls.

This method is shown in Fig. 10. Anchor bolts, *a*, *a*, are embedded in the wall at the time the brick or stone is laid, then when the wall is complete and the cement set, the sluice gate is bolted to the wall with a layer of cement mortar packed between the flange and the mason work. A wooden templet with holes bored in the same relative positions as in the sluice gate frame is used to hold the bolts in place while being bedded in the masonry.

Sluice Gate Bolted to Pipe—When sluice gates are to be connected to iron pipe they generally are bolted to a flange of the pipe as shown in Fig. 11. Instead of this method, however, the sluice gate is sometimes cast with a spigot end so that the spigot can be calked to a cast-iron pipe hub.

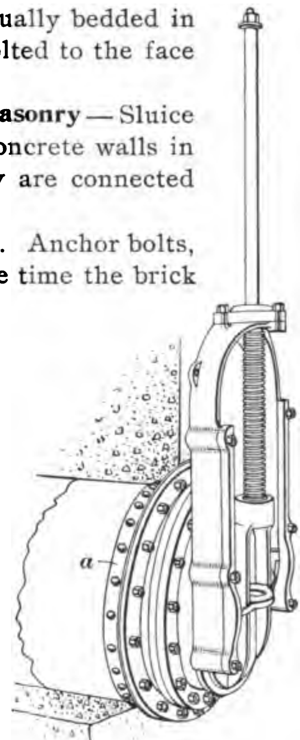


Fig. 11

Gate Operating Devices—Small sluice valves, also gate valves, are easily operated by hand without the aid of mechanism. In large plants, however, the sluice valve or gate valve, as the case might be, is extremely heavy and sometimes is located at a very low elevation. Under such conditions a stem is generally extended to above the grade line where it terminates in a gate house, and a capstan is provided direct connected to the stem of the sluice or gate, so that the gate or sluice can be controlled from the gate house. Fig. 12 shows one form of capstan for medium size gates. This device is provided with a cast-iron hand-wheel which has three sockets, *a*, *a*, *a*, cast in the hub, into which capstan bars can be inserted to start the gate when closed, or to firmly seat the gate when closing. After starting the gate with the capstan bars, it can easily be raised with the hand-wheel alone.

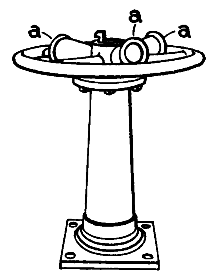


Fig. 12

Large sluice gates, that are too heavy to be easily operated by a hand wheel, are operated by some modification of the geared capstan shown in Fig. 13, or by electric motors or hydraulic lifts.

Gate Valves—The flow of sewage through pipes at purification works, is controlled by sluice gates when the gates can be placed in a tank or manhole; when, however, the valve must be placed on a pipe, a gate valve similar to the one shown in Fig. 14 is generally used. Valves of this type can be had in any size up to 10 feet in diameter and with or without spur gear for opening and closing the gate. The smaller sizes are not provided with gears, as the gate can be operated direct from a hand wheel or by means of a key with cross arms operated by two or four men. A gate of this description can be located in a valve pit and

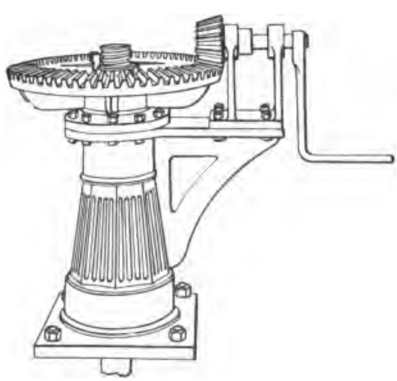


Fig. 13

operated by means of a cross-arm key, or the stem, *a*, can be extended to a gate house above, from which place it can be operated by means of a capstan, an electric motor or a hydraulic motor.

DOSING APPARATUS

Miller Automatic Siphon—A Miller siphon of slightly different design from the one shown in connection with the septic tank, is illustrated in Fig. 15. The action of this siphon is as follows: Assuming that the trap is full of water, then, as sewage flows into the tank it gradually rises above the bottom of the bell, thus confining the air between the mouth of the bell and the surface of the water in the trap. As the head of water in the tank increases, it compresses the air in the long leg of the trap, thus gradually forcing out the water until a point is reached when the air is about to escape around the lower bend.

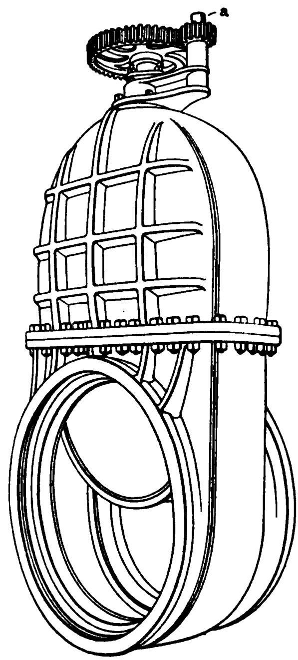


Fig. 14

The difference in water level in the two legs of the trap is at all times equal to the head of water above the level of water in the bell; consequently, when the water in the long leg of the trap is depressed to the point shown in the illustration, the siphon is about ready to operate, and any further increase in the depth of water in the tank will force the air around the lower bend, and in its upward rush the air will carry with it some of the water in the short leg of the trap, thus destroying the equilibrium of the two columns of water and bringing the siphon immediately into full action. The water is thus drawn out of the tank to the bottom of the bell, the siphon broken by the admission of air through the snift hole, *a*, and the siphon is again ready for action.

The depth of sewage that will accumulate in a tank, before the siphon is brought into action, depends on the length of the short leg of the trap. The head, *b*, from the level of liquid within the bell to the surface of liquid in the

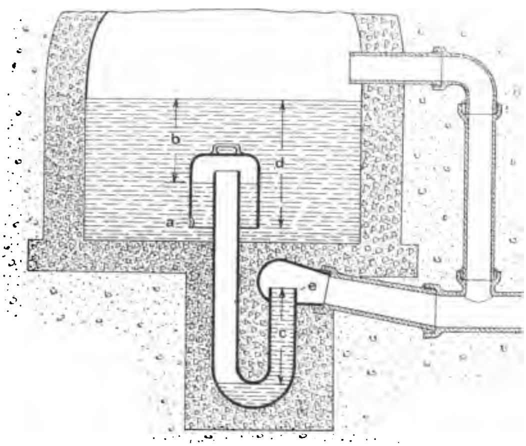


Fig. 15

tank will equal the distance, *c*, from the bend of the trap to the surface of the outlet pipe. In calculating the quality of liquid that will be discharged from the tank at each operation of the siphon, the head or depth, *d*, must be assumed, as the liquid will be lowered at each discharge to the top of the snift hole.

To find the quantity of the discharge, therefore, or to calculate the size of the tank required to discharge a given quantity of sewage, multiply the area of the tank in square feet by the head, *d*, in feet. The product will be the quantity in cubic feet discharged at each operation of the siphon. The illustration shows the outlet leg of the trap discharging into a pipe drain. Instead of this arrangement, however, the siphon can discharge into a special dosing tank connected to a drain, or into an open flume. The only requirement in any case is that the overflow edge of the short leg project into the tank, pipe, or flume, so that the water carried out by the air can instantaneously escape. If the discharge mouth were formed as an ordinary bend, the siphon would not operate, because the heaved-up water would have no means of instantaneous

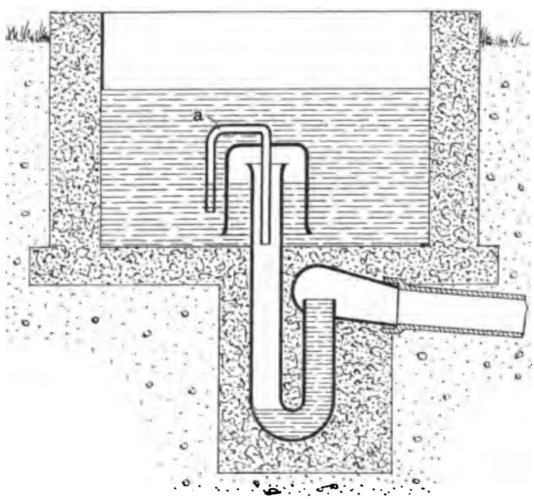


Fig. 16

escape, and therefore the equilibrium of the two columns of water would not be sufficiently upset.

A Miller automatic siphon, similar to the one shown in the illustration of a septic tank, is shown in section in Fig. 16. This siphon differs from the one shown in Fig. 15, principally in having a vent pipe *a*, instead of a snift hole in the bell. By using a vent pipe a greater depth of sewage can be accumulated in the dosing chamber before the siphon is brought into action. This is due to the fact that the air in the long leg of the siphon can escape until the sewage in the tank rises high enough to seal the mouth of the vent pipe, which, while sufficiently large to permit the escape of air from the confined space, is not large enough to break the siphonic action, when the mouth is unsealed, before the sewage is lowered to the mouth of the bell. This type of siphon or some of its modifications is the one most extensively used in purification plants.

Capacity of Siphons—The size of siphon required to empty a sewage tank depends somewhat upon the place of discharge. For instance, if the effluent is to be discharged intermittently into a stream, lake, ravine, or into tide water, the length of time required to empty the tank is of less importance than when the effluent is to be discharged onto a filter bed or into contact beds. When sewage is discharged direct into water, a slow discharge will be found conducive to greater dilution, and under such conditions a small siphon no doubt would be preferable. In the case

TABLE V—CAPACITY OF MILLER AUTOMATIC SIPHONS

Diameter of Trap in Inches	Rate of Discharge per Second	
	Cubic Feet	U. S. Gallons
3	.2	1½
5	.65	5
6	1.00	7½
8	2.00	15

of sand filters on the other hand, the beds should be flooded in a comparatively short period of time to bring all parts of the filter into service simultaneously; and in the case of contact beds the tank should be filled in a certain period of time, usually two hours.

When designing a purification plant, the places of disposal should be considered and the quantity of liquid to be discharged in a given time calculated; then, if a Miller siphon is to be used, the required size can be found in the following table of manufacturers' ratings, or the size can be calculated by the formula given on page 85.

Rhoads-Miller Automatic Siphon — An automatic siphon of the Rhoads-Miller type is shown in Fig. 17. The action of this siphon depends on the sudden releasing of the compressed air confined in the long leg of the trap, between the water in the bell, *a*, and the water in the deep-seal trap, *b*. Assuming that the deep-seal trap, *b*, and the blow-off trap, *c*, are sealed with water, the operation of the siphon will be as follows: Sewage will flow into the tank until it covers the mouth of the bell, *a*, and the outlet,

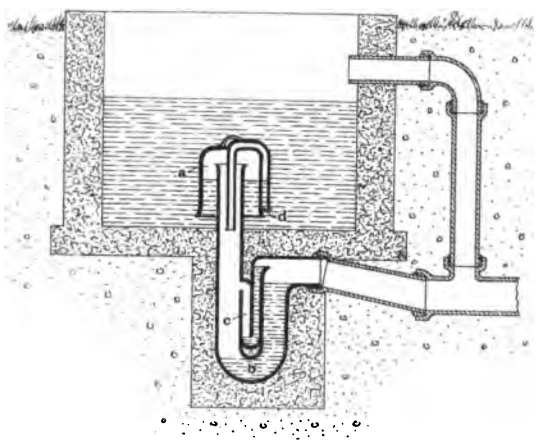


Fig. 17

d, to the vent pipe. The air confined in the space between the deep-seal trap and the bell will then become compressed in proportion to the head of water in the tank, until the pressure becomes great enough to force the water in the blow-off trap down to the bend or dip, *b*, that forms the seal. Any greater head of water will then force the water out of the blow-off trap, thus releasing the air from the confined space which immediately fills with water, thereby bringing the siphon into action. When the sewage has been lowered to the mouth of the pipe, *d*, air will enter through this pipe and break the siphonage.

Plural Alternating Siphons—Alternating siphons are used to discharge the effluent from a septic tank onto two or more filter beds in rotation, so that, between dosings, each bed will have a period of rest during which the interstices of the sand or other filtering medium can fill with air and thus provide oxygen for the reduction of the organic matter that is contained in the filter.

A plural alternating siphon of the Miller type is shown in Fig. 18.

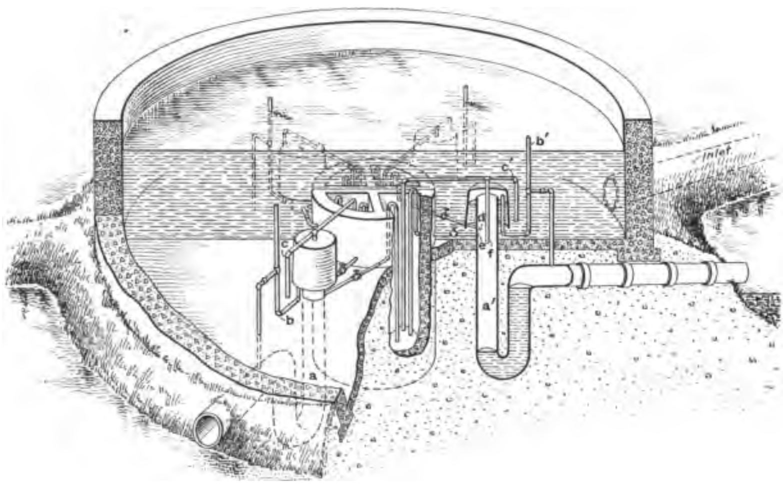


Fig. 18

This apparatus is designed to automatically discharge the contents of a dosing chamber onto several filter beds in rotation. The operation of the apparatus is based on the principle that, when two or more siphons are set at the same elevation in a tank, that siphon which contains the least water will discharge first; and, when installing an alternating siphon, the traps are cross-connected in such a manner that they all, with the exception of the one next to discharge, will refill at every discharge of sewage from the tank.

The alternating siphon is operated as follows: Assuming that the main traps, *a-a'*, and the blow-off traps, *b-b'*, are sealed, and the wells in the center filled with sewage, then when sewage rises above the mouth of the pipes, *c-c′*, the air in the traps will become confined and will be compressed in proportion to the rise of sewage in the tank. As the air becomes compressed it forces the sewage down in all the traps, as shown in the section, until air is about to escape around the dip of the trap which holds the least sewage. If, in the present example, the siphon shown in section were the first to discharge, sewage in flowing past the mouths of the pipes, *d*, *e* and *f*, would siphon sewage out of the two wells to which the pipes are connected, while the other two wells would remain full. These full wells would then fill the two traps to which they are connected, while the trap shown in section would immediately refill after siphoning, thus leaving the seal of the remaining trap weakened so it would be the next to operate. Any number of siphons can be connected up to operate automatically in rotation and they can be set in rows or in a circle. It is not necessary, however, to provide wells and cross connect the traps when only two siphons are in battery, and when only three Miller siphons are to alternate, the wells can be omitted although the traps must be cross-connected to one another.

Barbour Rotation Dosing Apparatus—A rotation dosing apparatus, designed by F. A. Barbour, of Boston, Mass., and successfully used in a number of plants, is shown in Fig. 19. In this apparatus a Miller siphon is used, which discharges into a distributing stand-pipe controlled by a revolving gate. Briefly, the operation of the apparatus is as follows: Assuming that the main trap, *a*, and the blow-off trap, *b*, are filled with sewage, which has raised above the mouth of the bell, *c*, and of the vent pipe, *d*, thus confining and compressing the air in the space, *e*, and blow-off pipe, *f*, then as sewage flows into the dosing chamber it raises the float, *g*, to the level shown in the illustration, thus by means of the ratchet and pawl, *h*, and bevelled gear, *i*, turning the rotary gate, *j*, in the stand-pipe until the opening is opposite the outlet to the next field to be flooded. It is obvious that the distance the float, *g*, is permitted to rise will depend on the number of outlets in the stand-pipe, a greater distance being necessary when only two or three openings are provided than when there are outlets to five or six filter beds. A stop is therefore provided, so that the float cannot rise higher than a certain level while the ratchet permits the float to return to its place at the bottom of the tank without disturbing the gate, *j*.

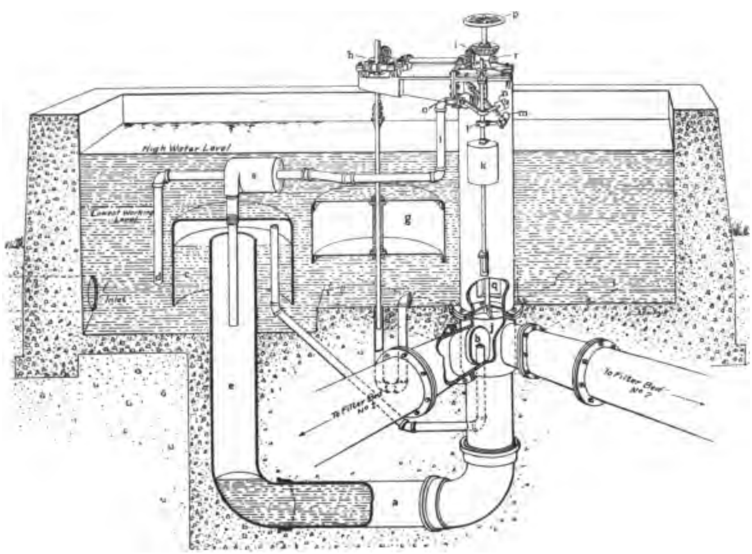


Fig. 19

In order to make the discharges automatic, and so they can be regulated to any desired height of sewage in the tank, a second float, *k*, is provided and is attached to a rod which works in guides. Above this float and attached to the rod is a tappet arm, *l*, which, when the float reaches a certain level in the tank, trips the lever, *m*, which in turn raises the weighted arm, *n*, that closes the escape valve, *o*. Air is thus released from the space, *e*, which immediately fills with water and the siphon is brought into action. By having an adjustable float, *k*, the siphon can be made to operate at different levels, thus making it possible to change the size of the dose applied to the beds, a condition which is sometimes desirable, under different weather conditions, it being understood that in summer small doses more frequently applied at high rates, give the best results, while, during winter in cold climates, a larger dose is required to prevent freezing of the beds.

The gate, *j*, can be operated by hand, by turning it by means of the wheel, *p*. A sluice gate, *q*, with its handle, *r*, projecting above the stand-pipe, is provided so sewage can be drained from the tank without passing through the siphon. A separator, *s*, is placed in the air pipe to prevent any solid matter being carried to and interfering with the operation of the escape valve, *o*. The blow-off trap, *b*, is provided to insure the discharge of the larger siphon at a predetermined head, regardless of whether or not the auxiliary float and escape valve on the air pipe work.

INTERMITTENT FILTRATION OF SEWAGE

Principles of Intermittent Filtration—If organic matter be deposited on the surface of the ground, or buried in the upper layers of the earth, certain changes will take place in the organic matter, which in a short time will disappear leaving only a residue of mineral ash that is soft and greasy to the touch, and resembles in its composition the humus of loamy soil. The changes that take place in the organic matter are brought about by micro-organisms with which the upper layers of earth fairly teem, in many cases numbering millions of bacteria to each gram of soil. Most of the micro-organisms of the soil are aërobic and are found in greatest number near the surface, in the first half-inch layer. From this point they decrease in number with the depth, until at a point five feet from the surface they are so few, comparatively, that the purification effected below that depth is but slight. Tests made by the Massachusetts Board of Health to determine the comparative number of bacteria in a gram of sand at different depths in a filter bed give results shown in Table VI.

The same agencies that operate to reduce organic matter in the upper layers of soil are equally active in a filter bed made of sand. After a sand filter has been ripened by intermittent applications of sewage, for a certain length of time, if sewage is then applied to it intermittently at certain periods of time, and sufficient intervals are allowed between the applications for the interstices of the sand to be drained of water and thoroughly aërated, it will be found

TABLE VI—BACTERIA FOUND AT DIFFERENT DEPTHS IN FILTER BEDS

Distance from Surface	Number of Bacteria per Gram of Soil
0 to ½ inch	1,760,000
½ to ¾ inch	105,000
1¼ to 1½ inch	207,200
2 inches	60,200
3 inches	111,300
5 inches	63,400
8 inches	30,700
12 inches	34,100
19 inches	12,300
60 inches	4,100

that the organic matter in the sewage will disappear and that the effluent from the filter will be clear, odorless and sparkling. The changes which take place in a filter bed are not due to the mere mechanical process of straining, but are the result of a biological process similar to that in a garden soil. If the grains of sand are examined microscopically it will be found that they are enveloped in a thin film of gelatinous material in which are entangled the reducing micro-organisms. This gelatinous film is pervious to water and under its influence swells, so that when sewage is applied to a filter bed, the gelatinous films that cover the grains become saturated with water and, by swelling, fill the interstices between the sand grains, thus interposing a barrier against the passage of organic matter without greatly retarding the flow of water. The bacteria in the gelatinous film attack the organic matter intercepted by the film and quickly convert it to useful nitrites and nitrates.

It is now generally understood that the mechanical separation of any part of the sewage by straining through sand is but an incident, which under some conditions may favorably modify the result; but the essential condition is a slow movement of a thin film of liquid over the surface of the particles, and sufficient voids between the particles to allow air to be continually in contact with the films of liquid.

If the action of a filter were simply a straining process, the surface of the filter and the interstices between the sand grains would soon be covered with a layer of sludge that would clog the filter. As a matter of fact, when a filter is not overdosed, no such clogging occurs; on the contrary, sewage containing sufficient organic matter to fill the voids of the sand bed many times over has been applied to filters daily for years, with the result that in every case all organic matter has been reduced, and the effluent from the filters has been clear, sparkling, odorless, and pure as many drinking waters, while with the exception of a small amount of cellulose material, no organic matter can be found on the surface of the filters or in the interstices of the sand.

Furthermore, in filters that are constructed of coarse material, for instance of crushed stones about the size of a walnut, a purification of the sewage is effected although the interstices between the stones are too large for the filter to act as a strainer.

It is absolutely necessary for the successful operation of a filter that the application of sewage be intermittent, and that sufficient time be allowed between doses for the organic matter to be entirely reduced; otherwise the surface of the filter bed will become clogged with organic matter, which, in some instances, becomes very thick and so interwoven that it can be rolled up like a mat, while in other cases the organic matter becomes caked on the surface.

The intervals between which filter beds may be flooded depends to a great extent on the strength and freshness of the sewage and on the size of the filtering interstices. Less time is required between applications when treating a weak domestic sewage from a septic tank on a coarse sprinkling filter, than would be required between applications on an intermittent sand filter when treating a strong sewage that is comparatively fresh. For an intermittent sand filter, treating average sewage, in moderate climates, it seems best to divide the daily dose into four portions to be applied at equal intervals of six hours. During winter weather, however, in cold climates, it is better to apply larger doses at less frequent intervals to prevent freezing of the beds.

In sewage purification by intermittent filtration, two classes of bacteria are brought into action. When the filter beds are flooded with sewage, anaërobic bacteria become active in breaking down, gasifying and liquefying the solid matter, after which, when the water has drained from the bed, aërobic bacteria complete the process by nitrifying the ammonia compounds.

Example of an Intermittent Filter

An intermittent filter is simply a bed of sand, or other porous material, suitably underdrained to carry off the liquid that percolates through the filter bed. Where filter beds are artificially constructed they are generally provided with underdrains of tile, brick or stone. In some localities, however, sand in its natural bed is available, and when such sand fields have a low water table they may be successfully used without pipe underdrains, the liquid that percolates through the sand being allowed to follow the natural course of the ground water. It would seem a better practice, however, to provide underdrains even when the filter beds are made in natural deposits of sand. An artificial filter bed is shown in Fig. 20. In this illustration the embankments and bottom of the filter are puddled with clay, and the slopes and tops of the embankments are sodded with grass. The underdrains, *a*, are usually made of ordinary salt-glazed sewer pipe with the bottom portion of the hub broken or cut straight across so the lengths will have a firm bearing on the bottom of the filter. Around the underdrains the filter is filled to a depth of several inches with broken stone that will pass through a 2-inch ring; above the broken stone are placed layers of graduated sizes of stone and gravel to provide a base for the bed of sand, *b*, which usually is about 5 feet in depth. The graduated sizes of stone and gravel are interposed to prevent sand from falling down and clogging the underdrains, or filling the interstices of the stones so the effluent would have to force its way laterally to the drains.

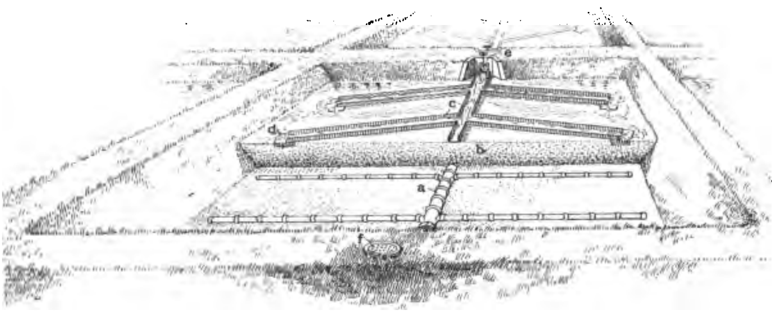
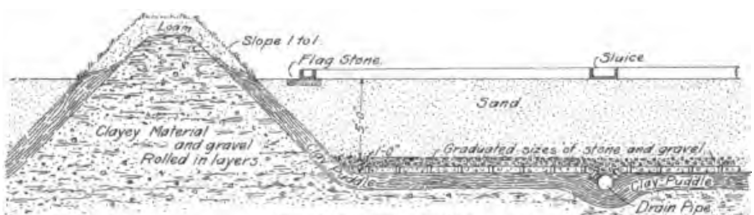


Fig. 20

On top of the sand bed, a system of distributing troughs, *c*, is provided to distribute as evenly as possible the flow of sewage to all parts of the filter, so that each section will have imposed on it an equal share of the burden. The surface of the filter bed is usually protected at the end of the troughs, at *d*, by paving with stones, flags or cement blocks, to prevent the sand being washed away; and sluice gates are provided as at *e* to control the flow of sewage to each bed unless each bed is flooded direct from a dosing tank through a separate and independent pipe, in which case the sluice gates are omitted. Where inlet pipes pierce the embankment, as shown at *e*, the slope should be protected by a bulkhead of brick, stone, or concrete, and the surface of the filter bed, at this point, should be paved with the same material. The cover to the manhole where the main effluent pipe from all the beds, and the main drain, *a*, from the bed illustrated, intersect, is shown at *f*. It is a difficult matter to lay down rules for the construction of filter beds that will be applicable to all cases. Illustrations showing the principles involved can be given, and the originality of the designer must then adapt to his use the method best suited to his purpose.

DETAILS OF INTERMITTENT FILTERS

Filter Basin—The materials of which filter basins are made depend much on the availability of various building materials in the locality where the purification plant is to be constructed, also on the type of filter bed. When sprinkling, or percolating, filters are to be constructed, an impervious flooring is provided and sometimes the system of underdrains and the filtering materials are placed on the flooring without being confined by walls or embankments. In the construction of intermittent sand filters, however, the filtering material usually is enclosed with a wall or with an earthen embankment that is impervious to water. It logically follows that in a locality where clay is plentiful, and sand, stone and cement expensive, puddled clay bottoms and embankments would be more economical than masonry walls and floors. Under such conditions, and in view of the further fact that a puddled clay basin will prove equally serviceable, this form of construction would



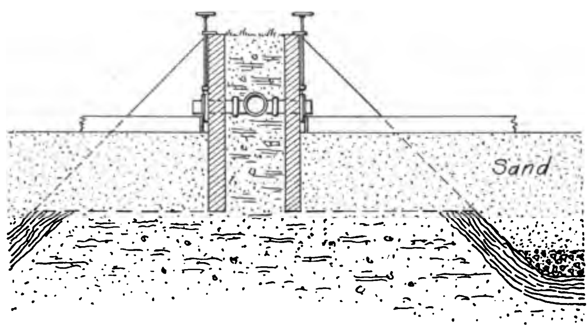
CROSS SECTION THRO' FILTER BED.

Fig. 21

doubtless be used; whereas in a locality where cement and sand are cheap, and clay expensive, a concrete flooring and enclosure would be preferable.

In laying a flooring and embankment of clay, greater solidity is obtained by laying alternate courses of clay and gravel than would be obtained by making the floor and embankments entirely of clay, while at the same time an equally tight basin is constructed.

A section of an intermittent filter showing the embankment, underdrains, stone, gravel, sand and distributing



SECTION THRO' GATE VALVES.

Fig. 22

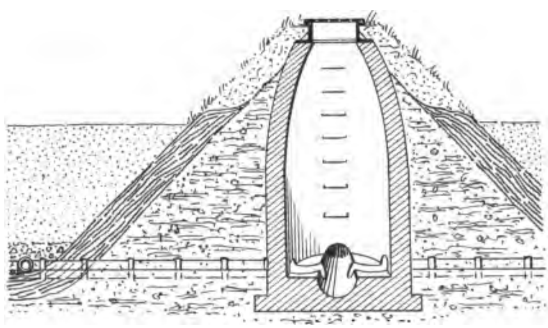
sluices is illustrated in Fig. 21. The embankments in this illustration are shown with a slope of 1 to 1. In some basins, however, the embankments are given less of a slope, being laid at from 1½ to 2 on the horizontal to 1 on the rise. The manner of building the embankments by rolling clay and gravel in alternate layers, the puddling of the basin with clay, the top dressing of loam, and the sodding of the embankments, are all clearly indicated in the illustration.

Where a number of filter beds are laid out together, however, at large purification works, the clay and gravel embankments are sometimes omitted and ordinary soil heaped up to form the basins; often when the soil is of a porous nature, puddling is also dispensed with and the underdrains laid on undisturbed earth. In fact, many modifications of the filter shown in the illustration are employed when designing filter beds, simplicity of design and economy of construction being the object desired.

A section through the gate or sluice valves controlling the flow onto filter beds is shown in Fig. 22, and a section through the manholes located at the intersection of the main drain from each filter bed and the main effluent pipe is shown in Fig. 23.

FILTER UNDERDRAINS

Systems of Underdrains—A system of underdrains



SECTION THRO' MANHOLE.

Fig. 23

either for an intermittent filter or for a sprinkling filter should be so proportioned that the frictional resistance will not cause unequal rates of filtration in different parts of the bed. Equal rates of filtration can be maintained by proportioning the size of the underdrains to the area they are to drain and the amount of liquid they are to conduct in a given time. As the rate at which filter beds are operated vary under different conditions, it is impossible to state a size of pipe to use for a unit area of filter bed. The size can readily be ascertained, however, by multiplying the area of filter bed to be drained by the proposed rate of filtration, then calculating the size of pipe that will safely conduct that quantity of water when running half full. The advantages of having drain pipes that are only half filled by the effluent from the surfaces drained are three-fold. In the first place, the frictional resistance is reduced to the minimum; in the second place, provision is made for a greater rate of filtration should a different preliminary treatment be subsequently devised that will effect a more complete purification of the sewage, thus permitting a greater rate of filtration; and lastly, that portion of the underdrains which is not filled with liquid will act as an air inlet to the bottom of the filter bed, thus providing a supply of oxygen for the aërobic nitrifying bacteria to complete the process of nitrification.

The size of drain required to conduct a certain quantity of water when running half full can be determined by the formula:

$$d = .234 \sqrt[5]{\frac{2q^2 l}{h}}$$

In which d=diameter of pipe in feet
q=cubic feet per second to be delivered
l=length of pipe in feet
h=head in feet

EXAMPLE—What diameter of pipe will be required to drain a section of filter bed containing 2,000 square feet of surface when the rate of filtration is 208,333 gallons per acre per hour; the drain being 100 feet long and laid at a grade of one foot in 100 feet?

SOLUTION—An acre contains 43,560 square feet of surface, or about 22 times the area contained in the 2,000 square feet of filter bed to be drained; consequently if the filter be operated at the rate of 208,333 gallons per hour, the quantity of sewage filtered by 2,000 square feet would equal $\frac{1}{22}$ of 208,333=9,469 U. S. gallons per hour. 9,469 U. S. gallons per hour equals .347 cubic feet per second.

Substituting the values in the formula,

$$d = .234\sqrt[5]{\frac{2 \times .347^2 \times 100}{1}} = .234\sqrt[5]{24} = .442 \text{ feet} = 5.3 \text{ inches. Answer.}$$

It is good practice to always calculate the size of pipe required to drain a filter bed. Much labor can be saved, however, and equally good results obtained in small plants by obtaining the size from Table VII. To use this table, calculate the quantity of fluid in cubic feet that must be removed per minute, double the quantity and find in the table the size of pipe corresponding to the grade that will safely conduct that quantity of water. The quantity of fluid is doubled so the size of pipe used will safely care for the effluent when running only about half full.

Example—What size pipe will be required to underdrain a section of filter bed 15 x 60 feet when the rate of filtration is 300,000 U. S. gallons per hour, and the drain is laid at a grade of 1 to 100 feet?

Solution—15 x 60=900 square feet=$\frac{1}{48}$ of an acre; $\frac{1}{48}$ of 300,000 U. S. gallons=6,489 gallons per hour equals 14.4 cubic feet of fluid to be removed per minute.

To find the size of drain pipe that when running half full will conduct that quantity of liquid, multiply 14.4 by 2, which gives 28.8, and find in Table VII (on following page) the size of pipe laid at a grade of 1 to 100 that will conduct that quantity of water. At the bottom of the column headed 5 inches, will be found that a 5-inch pipe laid at a grade of 1 foot to 100 feet will discharge 25.92 cubic feet per minute, and as that size of pipe comes the nearest to handling the required amount of water, when laid at the desired grade, it should be used.

If the drain be laid at a greater incline than 1 foot in 100 feet, a smaller size of pipe can be used. For instance, by a reference to Table VII it will be seen that a 4-inch drain laid at a grade of 1 foot in 30 feet has a capacity equal to a 5-inch drain pipe laid at a grade of 1 foot in 100 feet.

In proportioning an underdrainage system, the branch pipes are not reduced in size in proportion to their distance from the effluent outlet, but are extended in full size from the main collector to the end of the drain. Ordinarily there are but two sizes of pipe used for underdrains in any filter bed. The main conduit, which is proportioned to conduct the flow from the entire filter, is extended in full size across the filter, and the laterals or branch drains are proportioned to care for the flow from their respective drainage.

TABLE VII—CAPACITY OF DRAINS

Velocity in feet per minute (as determined by the formula $v=3{,}000\sqrt{\frac{h}{l}\times d}$) and discharge in cubic feet per minute (by the formula $Q=V\,A$) of drains laid at different grades when running full.

In which V=velocity in feet per minute
A=area of pipe in feet
h=head in feet
l=length of the pipe in feet
d=diameter of the pipe in feet

Diameter	2 Inches		2½ Inches		3 Inches		4 Inches		5 Inches		6 Inches	
Fall Ft. in Ft.	Velocity Feet per Minute	Discharge Cubic Feet per Minute	Velocity Feet per Minute	Discharge Cubic Feet per Minute	Velocity Feet per Minute	Discharge Cubic Feet per Minute	Velocity Feet per Minute	Discharge Cubic Feet per Minute	Velocity Feet per Minute	Discharge Cubic Feet per Minute	Velocity Feet per Minute	Discharge Cubic Feet per Minute
1 in 20	273	5.46	297	8.91	335	13.40	390	32.40	432	58.32	480	93.60
1 in 25	246	4.92	273	8.19	300	12.00	345	28.64	387	52 25	450	87.75
1 in 30	220	4.40	249	7.49	270	10.80	312	25.89	351	47.39	390	77.65
1 in 35	204	4.08	228	6.84	250	10.00	288	23.80	324	43.74	360	70.20
1 in 40	192	3.84	216	6.48	237	9.48	272	22.68	306	41.31	330	64.35
1 in 45	180	3.60	201	6.03	222	8.88	255	21.16	288	38.88	315	61.42
1 in 50	174	3.48	192	5.76	210	8.40	243	20.17	272	36.72	300	58.50
1 in 60	153	3.06	174	5.22	190	7.60	216	17.98	245	33.07	270	52.65
1 in 70	144	2.88	162	4.86	177	7.08	204	16.93	229	30.91	252	49.14
1 in 80	135	2.70	150	4.50	165	6.60	198	16.43	210	28.35	214	45.63
1 in 90	129	2.50	144	4.32	156	6.24	180	14.94	201	27.13	222	43.29
1 in 100	120	2.40	135	4.05	150	6.00	170	14.11	192	25.92	210	41.16

Diameter	7 Inches		8 Inches		9 Inches		10 Inches		11 Inches		12 Inches	
Fall Ft. in Ft.	Veloc-ity	Dis-charge	Veloc-ity	Dis-charge	Veloc-ity	Dis-charge	Veloc-ity	Dis-charge	Veloc-ity	Dis-charge	Veloc-ity	Dis-charge
1 in 20	510	135.15	540	189	573	252	620	335	690	455	750	585
1 in 25	480	127.20	480	168	510	224	540	292	570	376	600	468
1 in 30	438	116.07	450	158	471	207	510	275	520	343	540	420
1 in 35	390	103.35	408	143	441	194	456	246	480	316	510	397
1 in 40	363	96.19	390	137	411	180	432	233	450	297	480	374
1 in 45	342	90.63	360	126	390	172	405	218	430	283	450	351
1 in 50	327	86.65	345	120	363	160	390	210	410	270	420	327
1 in 60	288	76.32	309	108	330	145	345	186	360	238	390	304
1 in 70	270	71.55	280	98	306	135	324	175	340	224	360	280
1 in 80	252	66.78	270	94	294	123	309	167	325	214	330	257
1 in 90	240	63.60	258	90	273	120	285	154	300	198	315	245
1 in 100	221	58.56	245	86	258	114	270	146	288	190	300	234

NOTE—To determine discharge in U. S. gallons multiply cubic feet by 7.5.

areas, and are extended full size from the main conduit to near the filter walls.

The drainage system for a filter bed is shown in Fig. 24. In this system, the drains are made of ordinary salt-glazed sewer pipe, such as are used for house sewers, with the exception that the bottom portion of the hubs is cut off straight across and flush with the bottom of the pipe, so the lengths will have a firm bearing on the floor of the filter. The lengths of pipe forming the system of underdrains are laid with a space of 1 inch between their ends to provide openings for the effluent to enter the drains. The ends of the laterals in some filters are turned up and extended to above the water line in the filter, so that air will have free access at all times to the underdrains. This practice is not adhered to in all intermittent filters, but in sprinkling filters and in contact beds this or some other means of admitting air to the underdrains is advisable.

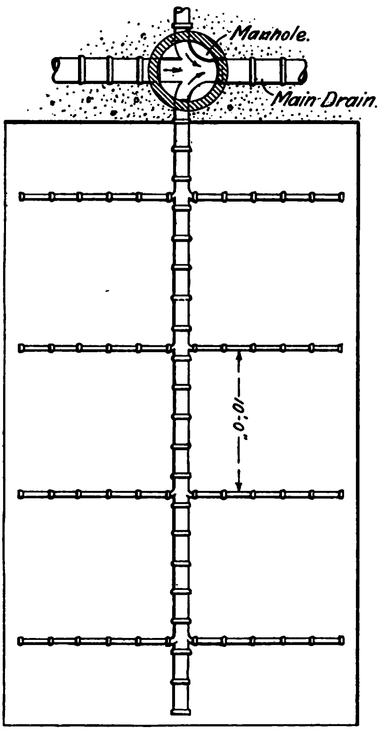


Fig. 24

The branches in an underdrainage system are proportioned according to the distance they are spaced apart and the area they drain. Ordinarily, pipes 3 inches, 4 inches, 5 inches and 6 inches in diameter are used, and they are spaced from 10 feet to 20 feet apart, according to the size of pipe and the area they drain. The grade at which underdrains are laid depends on the slope given to the bottom of the filter. Generally, the filter bottom is laid with a grade of 1 foot in 80 to 100 feet and the pipes forming the underdrainage system are laid on the floor of the filter.

Tile Pipe for Underdrains—A tile pipe such as is used for an underdrainage system in an intermittent filter is shown in Fig. 25. This is simply an ordinary salt-glazed sewer pipe with part of the hub, where it rests on the floor of the filter, broken away.

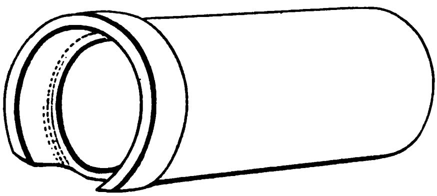


Fig. 25

Brick Underdrains—In some filters, in place of using pipe underdrains, drainage is provided by constructing a floor of two courses of common bricks. In this system of underdrainage, an illustration of which is shown in Fig. 26, the first course of bricks is set on edge and spaced about two inches apart, while the bricks in the second course are laid flat to provide a floor for the filtering material to rest upon. All of the bricks are laid with spaces between to provide drainage openings to the channels below formed by the lower tiers of bricks.

Perforated Tile Underdrains—A type of underdrain tile extensively used in the construction of slow-sand filters for water purification, and used to a limited extent in the construction of sewage filters, is shown in Fig. 27. The units making up this system of underdrainage are laid with open joints and cover the entire filter bottom. Both perforated tile underdrains and brick underdrains are more expensive than ordinary sewer pipe underdrains, and for intermittent filters possess no advantages that under ordinary conditions would justify their use. In sprinkling filters however they prove very satisfactory, as by their use a greater degree of aëration of the bed can be maintained.

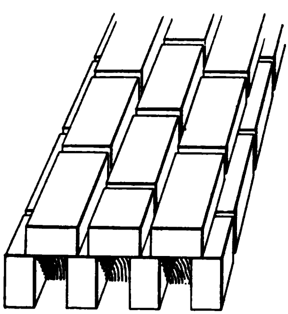


Fig. 26

Split-Tile Drains—Underdrains for sprinkling filters are generally made of split-tile, similar to that shown in Fig. 28. When this type of drain is used the tile are laid with open joints, as in the case of tile pipe, but unlike the system of tile pipe drains, which only has branches leading to a main line, when split tile drains are used, the filter bottom is completely covered with the tile which are laid with their longitudinal axes in the direction of the flow of the effluent. The effluent follows the slope of the floor to a collector located either at the center or at one side of the filter basin. The bottom edges of split-tile drains are scalloped to provide spaces for the effluent to enter the drains. The tiles are bedded firmly on the floor of the filter and the rows are placed at least one inch apart.

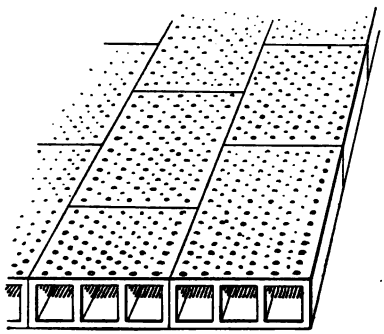


Fig. 27

FILTER BEDS

Filtering Materials—Many materials have been successfully used in filter beds for the filtration of sewage. Among the materials most commonly employed are crushed stone, coke breeze, coal, cinders, gravel and sand. The selection of a filtering material depends to a great extent on the type of filter to be constructed, the rate of filtration and degree of purity desired, and the availability of the different materials where the purification plant is to be constructed. Sprinkling filters are generally operated at a much higher rate than are intermittent filters, consequently a coarse grade of material

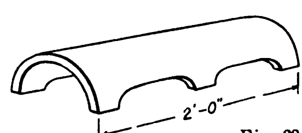


Fig. 28

must be used to permit the rapid flow of sewage through the bed and at the same time provide space for aëration of the effluent. The coarser the filtering material and the higher the rate of filtration, the less purification is effected, consequently when a great degree of purity is desired, intermittent filters constructed of sand will be found the most effective. Sand is the only material used for an intermittent bed.

The principal requirement of a filtering material is permanency; any material that will soften or disintegrate with use is unsuitable for such purposes, for after a time the disintegrated material will settle down and become compact in the bottom of the bed, thus clogging the filter.

SAND

Classification of Sands—Sand is composed principally of quartz grains and hard silicates that have been finely granulated by glacial action, or by the attrition of running water. Sand can be obtained from the sea-shore, from river beds or from sand banks. As it occurs in its natural bed, sand frequently is mixed with clay, loam, vegetable matter, limestone, or with other foreign substances, any of which, except limestone, should be removed before the sand is suitable for filtration purposes.

If sand containing clay or loam is used, the clay or loam will cement the grains of sand together and cause subsequent clogging of the filter. Vegetable matter is also objectionable when mixed throughout the entire depth of the filter bed. If the vegetable matter were only in the top layer of sand it could be reduced, as is all organic matter on the filter. Clay, loam and vegetable matter can be removed from sand by washing, and if sand that is free from such impurities cannot be obtained from sand bank, river bed or seashore, it should be washed before being used.

Sand containing limestone is objectionable as a filtering material for a water supply on account of imparting a certain degree of hardness to the water. For sewage purification, however, particularly in a soft-water region, a moderate degree of limestone in the sand instead of being harmful is actually beneficial, as it provides a base for the conversion of nitric acid into nitrates. The presence of limestone in sand can be determined by wetting the sand with hydrochloric acid; if it gives off a gas, the reaction indicates the presence of lime, the amount of which can be judged by the amount of gas given off and the appearance of the sample after the test. Sand taken from a sand bank usually has sharper and more angular grains than sand which has been exposed to the action of water. Sea-shore and river-bank sand usually is composed of grains that are more or less spherical and the angles of which are rounded by attrition. All other conditions being equal, a sharp angular sand is better for filtration purposes than is a smooth, round sand.

Uniformity Coefficient of Sand—It is obvious that all grains in a bed of sand are not of equal size, and this lack of uniformity must be taken into consideration when estimating the capacity of filters and the quality of the effluent likely to be obtained.

It was found by the Massachusetts Board of Health in the experiments they conducted at Lawrence, that, within certain limits, a sand in which the grains are of different sizes, will be more effective than will a sand with grains of uniform size. This increased effectiveness can be explained by the fact that in a sand of various sizes the sewage must pass around the larger grains and between the smaller openings of the finer particles that pack in between the larger sand grains. In other words, the interstices between the grains are smaller when a mixed sand is used than when the grains are of uniform size. It follows, therefore, that the finer portion of the sand determines the value of the sand for filtration, and the nearer to the right proportion this fine sand is to the whole, the nearer perfect is the sand. The proportion of coarse to fine sand is known as the uniformity coefficient.

That is, the uniformity coefficient of a sand is the ratio of the size of grain which has 60 per cent. finer than itself to the size that has 10 per cent. finer than itself.

In intermittent filter beds the uniformity coefficient should be as low as possible, and if sand of the right quality cannot be obtained in the locality, it may be worked over until the uniformity coefficient is about right; if the uniformity coefficient be too high, sufficient of the larger grains or pebbles can be removed by screening to reduce the uniformity coefficient to the desired standard. If, on the other hand, the sand be too fine, the finer particles can be removed by washing the sand, thus raising the uniformity coefficient.

Effective Size of Sand—In speaking of the size of sand the effective size is referred to. The effective size of sand is such a size of grain that 10 per cent. of the sand grains are smaller and 90 per cent. of the sand grains are larger than the size of grain that is known as the effective size. In practice the effective size of sand is found by sifting a definite weight of sand, usually 100 grammes, through a set of nested sieves, and, beginning with the catch on the bottom sieve, adding the sand that passed through the bottom sieve and the catch on each succeeding sieve until 10 per cent. or 10 grammes of the entire amount of sand is collected. The size of sieve that holds back the 90 per cent. and allows the 10 per cent. to pass through, determines the effective size which usually is expressed in millimeters.

The effective size of sand best suited to filtration purposes will be found not smaller than 0.20 millimeter in diameter, nor larger than 0.35 millimeter in diameter. Sand having an effective size of 0.26 millimeter has been found very favorable for the intermittent filtration of sewage. In measuring the size of sand, it is assumed that the grains are spherical and possess a diameter that is equal to the cube root of three axes, one of which is the longest and the other two taken at right angles to the longitudinal axis and at right angles to each other.

Calculating the Effective Size of Sand—The effective size of sand, also the uniformity coefficient, is found in the following manner: It will be assumed that 100 grammes of sand have been passed through a set of nested sieves and the catch of the various sieves weighed and found to be as tabulated below.

As the effective size of sand is that of which 10 per cent. by weight is smaller, and 90 per cent. larger, than itself, the effective size in this case must lie between the 0.1 per cent. that passed the .27 millimeter screen and

Number of Sieve	Size of Screen in Millimeters	Percentage of Sand Passing each Sieve
100	0.16	0.0
80	0.19	0.0
60	0.27	0.1
40	0.46	13.5
20	0.88	96.0
16	1.16	99.4
10	2.04	99.6
8	2.74	100.0

the 13.5 per cent. that passed the .46 millimeter screen; by interpolation it will be found that the effective size is equal to .41 millimeter.

Calculating the Uniformity Coefficient of Sand—The uniformity coefficient of sand is found by dividing that size of sand of which 60 per cent. by weight is either equal to or less, by the effective size of sand.

In the present example, the size of sand of which 60 per cent. by weight is equal to or less, lies somewhere between the 13.5 per cent. that passed the .46 millimeter sieve and the .96 per cent. that passed the .88 millimeter sieve. By interpolation it will be found that the size of sand is equal to .69 millimeter, and .69 millimeter divided by .41 millimeter, which is the effective size of sand, gives 1.7 millimeters which is the uniformity coefficient of the sand.

If all the grains of sand in a filter bed were of absolutely the same size, the uniformity coefficient would be 1.

With most sands, however, the coefficient ranges from 2 to 3, while a uniformity coefficient of from 1.6 to 2.5 will be found suitable for sewage filtration.

Nested Sieves—A convenient set of sieves for screening sand is shown in Fig. 29. This set consists of eight sieves, the largest of which is about 2 inches in diameter, while the rest of the set are graduated in size according to the size of mesh, so that when "nested" together, the smallest sieve having the coarsest screen is at the top and the largest sieve having the finest screen is at the bottom of the set. Beginning at the top, the sieves are numbered 8, 10, 16, 20, 40, 60, 80 and 100 respectively.

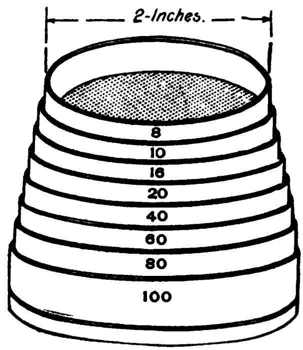


Fig. 29

The number of a sieve refers to the number of strands of wire per lineal inch. For instance, a No. 100 sieve has 100 strands of No. 40 Stubbs gauge wire per lineal inch, and as 100 wires cross at right angles, there are $100 \times 100 =$ 10,000 openings per square inch.

TABLE VIII—SIZE AND RATING OF SIEVES

Number of Sieves also Wires per Lineal Inch	Openings per Square Inch	Size of Openings in Millimeters
100	10,000	0.16
80	6,400	0.19
60	3,600	0.27
40	1,600	0.46
20	400	0.88
16	256	1.16
10	100	2.04
8	64	2.74

The size of opening in millimeters corresponding to the number or mesh of sieve can be found in Table VIII.

Standardizing Sieves—Sufficient dependence cannot be placed on the trade number of sieves to warrant their use without first being standardized. To standardize a sieve, a definite number of sand grains, those that are the last to pass through the sieve and which consequently more nearly correspond to the diameter of the mesh of the sieve, are taken. These grains are counted, and their sizes may then be determined either by micrometer measurements or by the weight of the particles. The sizes of the grains over 0.10 millimeter in diameter are more easily found by weight, while the sizes of grains smaller than 0.10 millimeter in diameter are more easily found by micrometer measurement. Micrometer measurements are made by taking the long diameter and the two middle diameters of the grains at right angles to the long diameter and to each other, as seen in a microscope. The three diameters are then measured by a micrometer screw, and, assuming that the grain is a sphere and possesses a diameter that is equal to the cube root of the three axes, the mean diameter is obtained by taking the cube root of the product of the three measured diameters.

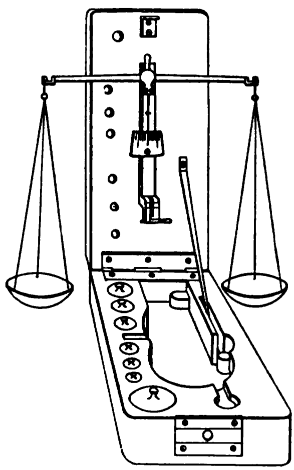


Fig. 30

EXAMPLE—What is the size of a sand grain that measures .30 millimeter along its long axis, .25 millimeter along its middle axis and .20 millimeter along its short axis?

SOLUTION—$.30 \times .25 \times .20 = .015,000$ and $\sqrt[3]{.015000} = .2466$ millimeter. Answer.

To determine the size of sand grains by the weight method, sift a certain quantity of sand, say 100 grammes, through a sieve, weigh the sand and count the grains. The weight of the sand divided by the number of grains will give the average weight of each grain. Having the weight of the grains the diameter can be found by the formula: *

$$d = .9 \sqrt[3]{W}$$

In which d=diameter of grain in millimeters
w=weight of grain in milligrams.

EXAMPLE—What is the diameter of a sand grain that weighs .02017 milligram?

SOLUTION—w=.02017 then,

$$d = .9 \sqrt[3]{.02017} = .242 \text{ millimeter. Answer.}$$

Having determined the size of grain that will just pass through the mesh of a sieve, the size of mesh in millimeters should be stamped on the outside of the sieve which is then standardized.

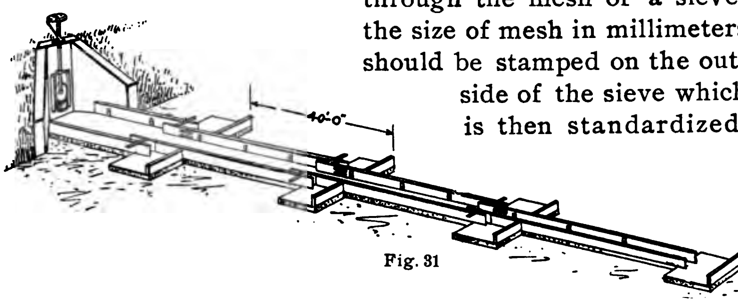


Fig. 31

Scales for Weighing Sand—A convenient pocket scale for measuring sand is shown in Fig. 30. Such an instrument is used by prospectors and others outside of a laboratory. It can be purchased from any manufacturer or dealer in chemists' supplies, and may be had in a leather or wooden case in a convenient size to carry in the pocket for field work. In ordering, assayers' weights should be specified.

SEWAGE DISTRIBUTORS

In order that sewage may be distributed uniformly to all parts of a filter bed, thus insuring an equal burden on all the filtering material, systems of distributors are employed which are so proportioned as to insure an even distribution of the sewage. A system of distributors that has

* Hazen. In deriving this formula, the specific gravity of sand was taken as 2.65, that being the specific of gravity sands used in the Lawrence, Mass., experimental filters.

been found very satisfactory in practice for distributing sewage to narrow beds, is shown in Fig. 31. These distributors may be made either of cement or of 2-inch planks. Hinged gates or sluice gates should be provided on distributors so the flow can be regulated or entirely cut off from any part of the filter bed which is being too heavily dosed. The surface of the filter bed when sewage is discharged from the distributors should be protected so the sand will not be disturbed at these points. Protection usually is effected by paving the surface with bricks, paving stones, or by putting a concrete or stone flag at the end of each branch from the distributor. Fig. 32 shows in

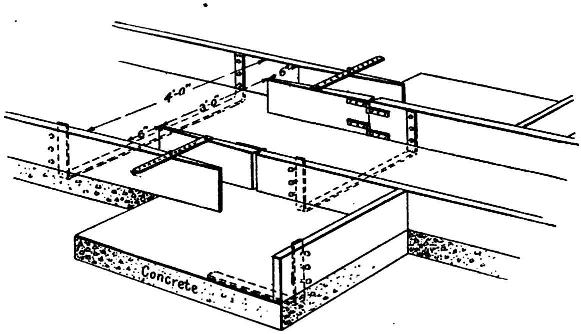


Fig. 32

detail the construction of distributing sluices. The bottom of the troughs is made of Portland cement concrete, mixed in the proportion of 1 cement, 3 sand and 5 stone or gravel. The flags are 6 inches thick, 8 feet long, and are laid with tarred paper in the joints between sections to provide for temperature changes. Iron bars bedded in the concrete are turned up at the edges as supports for 2-inch plank sides. Adjustable wooden gates are provided at the outlets to regulate the discharge if it is desired to throw unequal quantities on different parts of the bed. A 2-inch plank stop in front of each opening checks the velocity of the influent so it will not wash the surface of the filter.

A distributor system designed to distribute sewage to all parts of a filter bed is shown in Fig. 33.

In small or narrow filter beds, the distributor shown in a previous illustration will be found satisfactory, but in filter beds of large area branch distributors are desirable to prevent over dosing of part of the filter, while the rest of the bed remains undosed. It will be noticed that the main sluice is decreased in size as branches are taken off, so as nearly as possible to distribute the fluid equally over the entire filter surface.

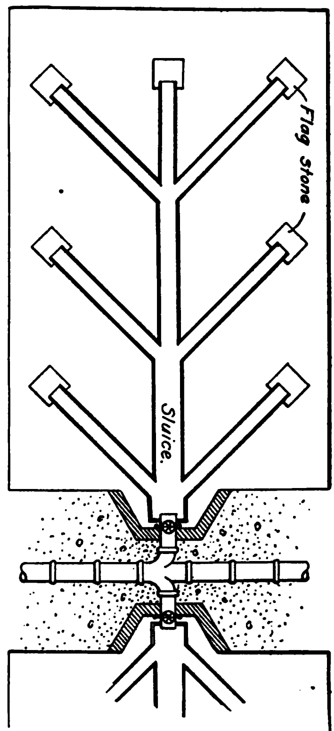


Fig. 33

A section of a cheaper type of distributing sluice is shown in Fig. 34. This distributor is made entirely of wood, and discharges the sewage onto the beds through scuppers in the sides of the sluices. Passing beneath the distributors at certain intervals are wooden cross-pieces, to which the bottom of the distributor is securely fastened to prevent warping. The cross-pieces extend beyond the sides of the distributors a sufficient distance to catch the flow of sewage from the scuppers and thus prevent surface washing of the sand bed. Shear gates are provided at all branch sluices to divert the flow of sewage from the main distributor into the branches. Wood is not an ideal material for distributing sluices. If a soft wood suitable in other ways for this purpose is used, the distributors are liable to become saturated with sewage and give rise to objectionable odors. The element of cheapness, however, will recommend this type of distributor, and when the purification plant is removed a sufficient distance from highways or habitations the additional odor from saturated sluices will hardly be noticeable.

Proportioning Distributors—In designing distributors for intermittent filters, they should be so proportioned that each branch will receive its share of the total flow and the main distributor, together with the branches, should be of sufficient size to discharge within a comparatively short time the entire contents of a dosing chamber. If the sewage is discharged onto the filter bed in a small stream, the liquid portion will sink into the sand within a short radius of the point of discharge, thereby overdosing that portion, while the rest of the area remains unused. When discharged in a large quantity within a few minutes' time, the sewage floods the entire filter area, thereby imposing on each part its proportionate share of the purification to be affected.

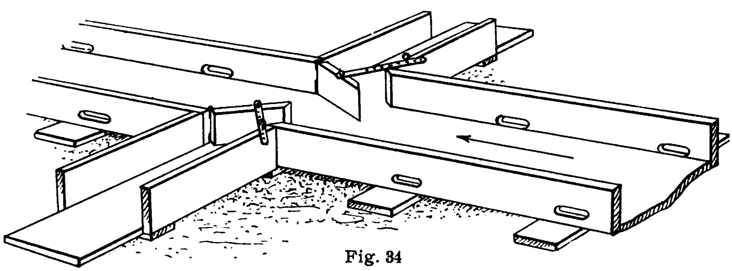


Fig. 34

The size of the distributing main for an intermittent filter depends on the quantity of sewage that is to be discharged in a given time. When a filter treats the effluent from a septic tank, the quantity of sewage to be conducted per minute will depend on the discharging capacity of the automatic siphon. Ordinarily, in large plants the siphon is proportioned to empty the dosing chamber in about ten minutes; so by dividing the capacity of the dosing chamber in cubic feet by ten minutes will give the quantity of sewage in cubic feet to be removed per minute.

Having the quantity to be removed per minute, a siphon having the required capacity can be found by the formula

$$d = .234 \sqrt[5]{\frac{q^2 l}{h}}$$

In which d=diameter of siphon in feet
q=cubic feet to be discharged per second
l=length of siphon in feet
h=head in feet

In this formula, the head *h* is taken as the difference in length between the long and short legs of the siphon. As a matter of fact, when the siphon is first brought into action, water flows through, due to the hydraulic head, and is not siphoned out, the siphon being brought into requisition only when sewage is lowered to the top of the overflow pipe within the bell.

EXAMPLE—What diameter of siphon will be required to discharge 8 cubic feet of sewage per second through a siphon 12 feet long, acting under a head of three feet?

SOLUTION—Substituting the values given in the example in the formula

$$d = .234 \sqrt[5]{\frac{64 \times 12}{3}} = .234 \times 3.031 = 1 \text{ foot diameter. Ans.}$$

Siphons are made in stock sizes that coincide with the standard sizes of pipe, and if the calculated size of a siphon is between two stock sizes, the larger size should be used.

The velocity of flow through an automatic siphon, such as is used for the discharge of effluent from a dosing chamber, is about 10 feet per second at the beginning of the operation and about 8 feet per second at the end of siphonic action; thus a large quantity of sewage at a comparatively high velocity is discharged in a short period of time on the distributors, which are laid at such a grade that the velocity of the flow in them seldom exceeds 4 feet per second and will perhaps average 3 feet per second. This decreased velocity should be taken into consideration when proportioning the distributor system, and the flumes made large enough to care for the sewage when flowing at a velocity of only 3 feet per second. When the quantity of sewage and the velocity of flow are known, the cross-sectional area of a rectangular flume to care for that quantity of sewage can readily be calculated by the formula

$$a = q \div v$$

In which a = cross-sectional area of distributor in square feet
q = quantity of sewage in cubic feet per minute
v = velocity in feet per minute

A flume proportioned to care for the sewage discharged by the siphon in the preceding example and solution, is calculated in the following example and solution:

EXAMPLE—What should be the cross-sectional area of a rectangular distributing flume to discharge 8 cubic feet of sewage per second at a velocity of 3 feet per second?

SOLUTION—Quantity of sewage to be cared for per minute $= 8 \times 60 =$ 480 cubic feet. Velocity of flow per minute $= 3 \times 60 = 180$. Substituting those quantities in the formula

$$a = 480 \div 180 = 2.7 \text{ square feet.}$$

2.7 square feet would be equal to a flume 4 feet wide by 8 inches in depth, which would be about the right proportion for a distributing flume. To provide against an overflow of effluent from the distributor should it be lined with ice or otherwise obstructed, the sides are usually made 10 inches high to allow a suitable margin of safety.

Size of Half-round Flumes—The size of a half-round flume required to conduct a given quantity of sewage can be calculated by the formula for finding the size of underdrain to discharge a given quantity of effluent when running half full. It is obvious that a pipe which will conduct 100 cubic feet of liquid per minute when running half full, will be equal in diameter to a half-round flume which will conduct the same amount when running full. Knowing the size of flume that will conduct a certain quantity of fluid when running full, using the next larger commercial size will allow a margin of safety that will be ample and will not overflow.

The size of pipe required to conduct a certain quantity of liquid, when laid at different grades and only half full, can be found by doubling the quantity of sewage to be conducted, and finding in Table VII the size of pipe that will care for that quantity.

AËRATORS

Object of Aërators—When sewage effluent passes from a septic tank, the fluid is devoid of oxygen. Up to this point, the process of decomposition has been an anaërobic one in which the nitrogeneous solids have been broken down into ammonic compounds, while the carbonaceous solids have been converted into carbon dioxide, alcohol, water, succinic acid and glycerine. The further process of sewage purification is an oxidizing one in which anaërobic bacteria are the oxidizing mediums which convert the ammonia and other compounds into useful nitrates. This part of the process is known as nitrification, and in order that the most favorable conditions be provided for the nitrifying organisms, the sewage should be saturated with air before being discharged onto the filter beds. Tests of sewage for oxygen show that before entering a septic tank the sewage contains from 0 to 5 per cent. of its capacity for air; upon leaving the septic tank the effluent is utterly devoid of air; after aëration the effluent contains about 75 per cent. of its capacity for air, and when applied to the filter beds about 40 per cent. of its capacity for air.

Aëration is not always provided for. When there is sufficient fall between the septic tank and the filter bed aëration is easily accomplished by discharging the effluent over a weir or by passing the effluent through an aërator, but when the fall is very slight, the only aëration that can be effected is that obtained by placing deflectors in the distributors to deflect the effluent from side to side, thus exposing as far as possible all particles to the atmosphere. Aërators are sometimes objectionable on account of the objectionable odors they release in the form of gas.

Example of an Aërator—An aërator that has been successfully used in practice is shown in Fig. 35. This apparatus consists simply of a standpipe, to which is attached a series of perforated baffle plates that break up the stream of liquid into thin films, small streams and drops, thus exposing all parts to the atmosphere. After flowing over or dripping through the plates, the liquid settles to the bottom of the aërator chamber and flows through an outlet into the collecting gallery. From here the liquid can be discharged through one of the sluice gates into the dosing chamber, or bi-passed around the dosing chamber either to the filter beds or to the sewer outfall. By opening the sluice gate which closes the effluent pipe, the septic liquid can be passed through to the dosing chamber or filter beds without passing through the aërator. A high degree of aëration can be effected by causing the liquid to overflow

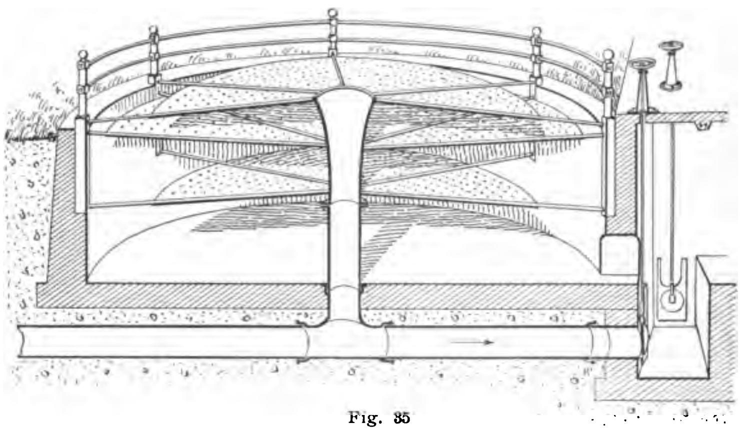


Fig. 35

a standpipe similar to the one shown in the illustration, but without the perforated baffle plates to break up the stream. Any form of sluice which will break up a column of water so as to expose it to the atmosphere in thin films, drops or spray, will prove an effective aërator.

Size and Capacity of Intermittent Filters—Filter beds usually are proportioned to the size of the dosing chamber in the purification plant, or the dosing chamber and filter beds are proportioned to each other, so that when the entire dose is discharged onto a filter the sewage will cover the surface of the bed to a uniform depth. Dosing tanks, however, are never made so large that they will require the construction of filters of greater area than one acre.

When the amount of sewage is large, it is found to be a better practice to decrease the size of the dosing chamber and filter areas and provide more beds. Reserve filtration capacity should always be provided so that the surface of one bed can be cleaned or raked while the others are in service. For this reason, a filtration plant of less than two beds is seldom constructed, even though one filter bed has sufficient area to care for the maximum flow of sewage.

The capacity of filter beds depends greatly on the strength and staleness of the sewage and on the effective size of the sand. Intermittent filters can be and in many localities are used to treat crude sewage, in which respect they are quite effective. The amount of crude sewage, however, that can be purified per acre of surface is comparatively small and more economic results are obtained where large quantities of sewage are to be purified by using intermittent filtration as a final process to follow some preliminary treatment.

Sewage is usually applied to the filter beds twice, three times or four times in twenty-four hours, although with very fine sand, sometimes only three doses a week is all that can be applied.

When treating crude sewage, the applications are made at twelve-hour intervals, while with septic sewage the period is cut down to from six to eight-hour intervals. With crude sewage, the dose is proportioned to cover the surface from 1 inch to 2 inches in depth, which being applied every twelve hours would be equal to a rainfall of from two inches to four inches over the entire surface every twenty-four hours. Crude sewage treated at this rate would be equal to the purification of from 50,000 gallons to 100,000 gallons per acre per twenty-four hours.

When treating septic sewage or very weak crude sewage, the fluid is applied in doses of from 2 inches to 4 inches in depth over the entire filter surface. It has been proposed in some instances, where sewage has been subjected to septic action and afterward passed at a rapid rate through a coarse sprinkling filter, to still further treat the effluent from the sprinkling filter by passing it at the rapid rate of 28 inches per day through intermittent filters. At this rate of purification, each acre of filter surface would purify 750,000 gallons of sewage per acre per twenty-four hours. The conditions are unusual, however, and if sewage be treated at that rate, provision no doubt would be required to scrape the surface of the sand, as in water filtration, and wash it for future use. Where land is expensive and the expense of labor offsets the interest that would have to be paid on land purchased for additional beds, or where sufficient area cannot be obtained, this practice might be advisable. It is doubtful, however, if sewage will often have to be purified to such a degree as to require intermittent filtration of the effluent from sprinkling filters.

For the treatment of septic sewage by intermittent filtration under ordinary conditions, a rate of 12 inches per twenty-four hours applied in three or four separate doses is about the maximum that can satisfactorily be purified. A rate of 12 vertical inches per twenty-four hours is equal to the purification of 326,700 gallons per day per acre of surface. At this rate of purification, and assuming a per capita consumption of 100 gallons of water per day, one acre of filter surface would be required for each 3,267 inhabitants. That is the greatest number of people, however, that one acre of filtration surface will care for, and when it is considered that sand which is suitable for that rate of filtration is not always obtainable; that reserve surface must always be provided so beds that have been overdosed can rest; that communities sometimes have their population temporarily increased ten to twenty per cent. by the influx of visitors; that ground water infilters into the sewers, and that provision should be made for growth in population, when all these conditions are considered, it is found that provision of one acre filter surface for each 2,000 inhabitants is about the greatest proportion.

Intermittent filtration of septic sewage at the rate of 500,000 gallons per acre per day has been successfully maintained under favorable conditions, and the tendency of the times is to increase the rate of filtration. However, in the present state of sewage purification practice, lower rates of filtration are safer and are sure to give more satisfactory results. To summarize the results obtained by intermittent filtration, it might be stated, that when treating crude sewage with clear fine sand, 30,000 gallons per acre per day can be purified to such a degree that the effluent is originally far superior to ordinary pure water, and the number of bacteria per unit volume is much less than in the pure water. When treating septic sewage on a similar bed, the same degree of purity can be obtained when operated at a rate of 60,000 gallons per acre per day.

With intermittent filters of clean, sharp, coarse sand treating crude sewage, 60,000 gallons per acre per day can be purified to such a degree that 97 to 99 per cent. of the organic matter and 99.9 per cent. of the bacteria will be removed, and the effluent will be colorless, generally clear and will possess little or no sediment. On similar filter beds 120,000 gallons of septic sewage per acre per day can be purified to an equal degree. On intermittent filters of coarse sand similar to the beds just described, 180,000 gallons of crude sewage or 360,000 gallons of septic sewage can be filtered per acre per day, removing 97 per cent. of the organic matter, 95 per cent. of the bacteria, and producing an effluent that will more than satisfy the conditions of any standard of sewage purification yet laid down. Such filters may occasionally require a period of rest or a working over of the top layers of the filtering material, while in some cases it might be necessary to remove and wash the top layer of sand.

Where crude sewage is treated by intermittent filtration, screens should be provided to hold back coarse, insoluble particles that would clog the surface of the filter bed.

A fair idea of the relation of size and uniformity coefficient of sand to size, frequency and size of dose of crude sewage that a bed will purify can be found in Table IX, which gives the actual quantities of crude sewage found the best to apply to beds of different sizes of material. The table will be found helpful not only in showing the actual amount of sewage that different sizes of sand will purify, but also in pointing out the best quantity to apply at a given time, the length of time which the bed should be allowed to rest and the size and frequency of doses.

The mechanical composition of the materials used in the various beds, the results of which are tabulated in Table IX can be found in Table X.

These materials are supposed to include the whole range of sands available for sewage purification.

TABLE IX—QUANTITY OF SEWAGE PURIFIED BY DIFFERENT SIZES OF SAND

(Massachusetts State Board of Health)

No. of Filter	Uni-formity of Coeffi-cient	Effective Size of Sand in Milli-meters	Depth of Filter Bed in Feet	Size of Dose		Number of Doses in One Week	Average Amount Applied Daily in Gallons per Acre
				Gallons per Acre	Per Cent. of Volume of Filter		
1	1.8	5.00	5	2,800	.17	500	200,000
2	2.4	.48	5	40,000	2.45	18	103,000
3	7.8	.35	4	70,000	4.37	6	60,000
4	2.0	.17	5	120,000	7.36	6	103,000
5	2.3	.06	5	140,000	8.60	3	60,000
6	2.3	.03	5	80,000	4.91	3	34,000
7	9	.02	5				

The data contained in Table X are plotted in diagram on next page, Fig. 36. "The lines representing the diameters are spaced according to the logarithms of the diameters of the particles, as in this way materials of corresponding uniformity in the range of sizes of their particles give equally steep curves, regardless of the absolute sizes of the particles, thus greatly facilitating a comparison of different materials. This scale also shows adequately every grade of material from 0.01 to above 10 millimeters, in a small space and without unduly extending any portion of the scale."

The height of curve at any point shows the per cent. of material finer than the size indicated at the bottom of the diagram.

It will be seen by the diagram that materials 2, 4, 5 and 6 have approximately steep curves, and by referring to the table it will be found that, while their effective sizes differ considerably, their uniformity coefficient is approximately the same. Likewise the curves of Nos. 3 and 7 are quite similar, and reference to the table shows that they have approximately the same uniformity coefficient.

(Mass. State Board of Health)

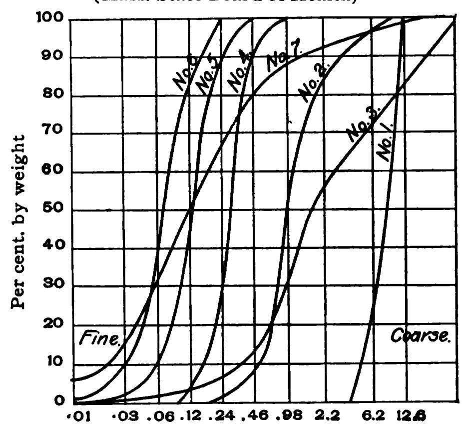


Fig. 36—Diameter of Sand in Millimeters

AIR AND WATER CAPACITY OF SANDS

Sharp grained sands having uniformity coefficient of less than 2, as ordinarily packed, have nearly 45 per cent. of open space or voids; and sands with coefficients between

TABLE X—MECHANICAL COMPOSITION OF SANDS AVAILABLE FOR FILTRATION

(Report of Massachusetts State Board of Health)

Diameter in Millimeters	¾ Per Cent.						
	No. 7	No. 6	No. 5	No. 4	No. 3	No. 2	No. 1
Finer than 12.6	99	....	....	....	83	100	98
Finer than 6.2	96	....	....	....	73	97	27
Finer than 2.2	92	....	....	....	57	85	....
Finer than .98	89	....	....	100	32	53	....
Finer than .46	80	....	100	91	13	7	....
Finer than .24	67	100	90	26	7	1.5	....
Finer than .12	51	85	43	3	4	....	....
Finer than .06	33	35	10	....	2	....	....
Finer than .03	16	10	2	....	.5	....	....
Finer than .01 organic	6	1	....	....	....	....	....
Effective size	.02	.03	.06	.17	.35	.48	5
Uniformity coefficient	9	2.3	2.3	2	7.8	2.4	1.8

3 and 2 usually have about 40 per cent. of open space. The voids are smaller in more mixed materials, and sands with uniformity coefficient of from 6 to 8 have only about 30 per cent. of open space, while in materials with extremely high coefficients almost no space is left. With water-worn, round-grained sands, the open space is from 2 to 5 per cent. less than for corresponding sharp grained sands.

(Mass. State Board of Health)

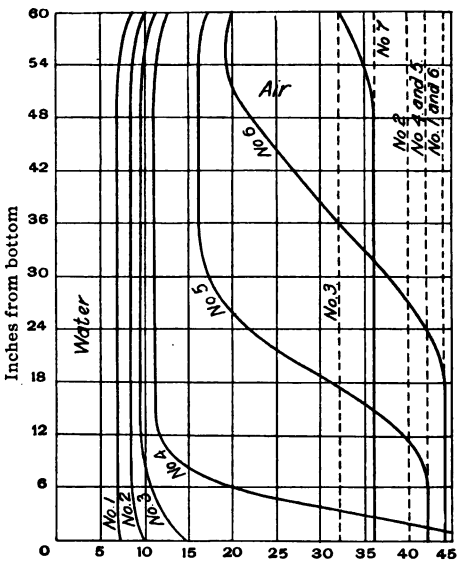


Fig. 37—Per cent. by volume

It is evident that the quantity of water within a sand bed can never exceed the volume of the voids, which is never over 45 per cent., and the water capacity is usually considerably less than the volume of voids. By the water capacity of a sand is meant the amount of water retained in the interstices after the bed has been thoroughly drained. The percentage of water remaining in beds of the materials described in the foregoing tables, can be seen by reference to the diagram, Fig. 37. This diagram gives not only the percentage, by volumes, of water retained at each 6 inches depth of the filter bed, but also the air capacity of the filter beds. The full lines represent water capacity and the dotted lines air capacities. It will be noticed that the curve of No. 6, which is the finest material listed, shows that the lower 18 inches of the bed are practically filled with water after the bed has been thoroughly drained, and that in No. 1, which is a very coarse material, the water drains out almost uniformly at all depths.

A classification of sands which gives the approximate size of sand for different classifications can be found in Table XI. This table gives the size of some particles which are much smaller than can be used for a filter bed. Sand having an effective size smaller than .03 millimeter in diameter is unfit for filtration purposes, while particles smaller than .01 millimeter are considered organic matter.

TABLE XI—CLASSIFICATION OF SANDS

Name	Diameter Millimeters	Inches Approximate
Fine gravel	2.0 to 1.0	.08 to .04
Coarse sand	1.0 to .5	.04 to .02
Medium sand	.5 to .25	.02 to .01
Fine sand	.25 to .1	.001 to .004
Very fine sand or dust	.1 to .05	.004 to .002
Silt	.05 to .01	.002 to .004
Fine silt	.001 to .005	.0004 to .0002
Clay	.005 to .0001	.0002 and under

Effect of Temperature on Intermittent Filtration—Temperature has a marked influence on the operation of an intermittent filter. As soon as frost begins to form on a filter bed a change can be detected in the chemical composition of the effluent; the free ammonia increases, the nitrates decrease and the organic matter, as shown by the albuminoid ammonia and by the oxygen consumed tests, also increases but not in the same proportion as the free ammonia. During extremely cold weather nitrification almost ceases and ammonia instead of nitrates is largely the end product of oxidation so far as nitrogen is concerned.

A low temperature does not affect to a great extent the first stage of purification, that which breaks down the organic matter into ammonia and carbonic acid, consequently while the purification effected is not chemically complete, almost as much of the organic matter is reduced as during the warmer months.

Frost has not so retardant an effect on the passage of sewage through a filter as would be expected; when sand is frozen, after draining, there still remains a certain percentage of open pores, and when sewage at a temperature of from 44 to 46 degrees Fahr., which is the average temperature of sewage in winter, is applied to the bed, the sewage easily finds its way through the open pores, thawing the frost as it proceeds. After the sewage has passed away there still remains in the bed a certain amount of water which again freezes, but is again thawed when the bed receives another application of sewage. Conditions might be such at times that temporarily a filter bed is made inoperative by ice. A filter bed of fine sand or one that is

Fig. 38

surface clogged to such an extent that sewage is retained on the surface for a considerable time in extremely cold weather, might have the sewage frozen on the bed, thus closing all pores. This condition however is rare in practice. Ordinarily, filter beds that are covered with snow and that are frozen to a depth of 36 inches, cause no trouble when sewage is applied, the effluent appearing in the drains within several hours after the application. As a matter of fact, snow seems to form a blanket over the surface of the filter, and when sewage is applied it flows under the snow which prevents evaporation and consequent chilling of the sewage. A further protection sometimes is provided for intermittent filters in cold climates by arranging the surface of the filter in furrows as shown in Fig. 38. Any ice that forms then bridges the furrows from ridge to ridge, allowing the sewage to flow beneath and percolate through to the underdrains. On the whole, it may be said that filters can safely be operated in climates where the mean temperature of the atmosphere in winter is not lower than 18 or 20 degrees Fahr.

Care of Filter Beds—A certain amount of material accumulates on the surface of filter beds, particularly when treating crude sewage. This material cannot properly be termed sludge, but is more of the nature of a dry cake of a stable and inoffensive character, which if not removed will clog the filter surface, thus reducing greatly the capacity of the filter. To remove this material the beds must be frequently raked and harrowed and occasionally plowed, spaded and scraped. It may be assumed as a fair average, that 8 cubic yards of sand and clogging material will have to be removed from a filter surface for each million gallons of crude sewage treated. After plowing or scraping a filter it should be carefully leveled, so that sewage will flow to all parts of the field.

SPRINKLING FILTERS

Principles of Sprinkling Filtration—The terms sprinkling filter and percolating filter are now indiscriminately used to designate a certain type of filter plant. In this work, the term sprinkling filter will be used as the better term to describe the type. In intermittent filtration, the name is derived from the method of applying sewage to the filter; by following the same system of nomenclature, the term sprinkling filter seems the better one to designate filters of coarse grained materials onto which sewage is sprayed through perforated pipes or sprinkler nozzles. The term seems the more suitable one when it is considered that, in any type of filter, the fluid percolates through the bed, and all might equally be classed as percolating filters.

Sprinkling filtration is a method of sewage purification in which either crude sewage or septic effluent may be applied in a fine spray, like drops of rain, to a specially prepared filter bed of coarse material. The material used in the bed of a sprinkling filter is too coarse to act as a strainer, consequently what purification takes place during the few minutes consumed by the sewage in passing through the filter bed, is due to bacterial action and the process is more of a biological than a mechanical one. Notwithstanding the fact that the process is mostly biological, and that the material of the filter bed is too coarse to act as a strainer, there is a certain mechanical action within a sprinkling filter which plays an important part in holding back the organic matter until bacterial activity has reduced it to a stable condition. When closely examined, the material of which the filter bed is composed, is found to be enveloped in a gelatinous film. This film is of such composition that it entangles and holds back anything brought in contact with it, which is not of sufficient weight and hardness to cut or tear the film. Owing to the depth of a filter bed, and the zig-zag course that sewage must take, even when flowing in large quantity through the filter, there is little or no part of the water and contained organic matter which is not brought into direct contact with the film on one or more pieces of filtering material; and, when the sewage is applied by spraying, the drops of sewage trickle down from filtering piece to filtering piece, depositing on each a portion of the organic matter it carries, until at last it leaves the bottom layer free from the organic matter it originally contained, but partly charged with other organic matter picked up in the filter bed, and which, while not entirely reduced, is in a non-putrefactive condition.

Owing to the coarseness of the filtering material and the manner in which sewage is applied, the sewage trickles in thin films over the surface of the filtering material in free contact with the air which fills the voids. The air, which amounts perhaps to five times the volume of sewage, is able under the most favorable circumstances to supply sufficient oxygen for the nitrifying organisms. It is assumed that five to ten volumes of air are required for each volume of sewage.

The process of sprinkling filtration is an aërobic one and must be carried on in the presence of air, with a plentiful supply of oxygen for the reducing bacteria to carry on the work of decomposition and nitrification. The interstices between the filtering material are full of air which is constantly replenished as fast as it is consumed by the circulation of air through the interstices of the filtering material, and by the application of sewage, which, falling like rain, not only is saturated with air, but carries more air along by momentum into the voids of the filter bed. This plentiful supply of air in a sprinkling filter-bed more or less completely oxidizes the organic matter without liquefying or gasifying it; consequently the effluent from a sprinkling filter instead of being clear like the effluent from an intermittent sand filter, contains, as a rule, a certain amount of flocculent organic matter. This organic matter, which is of a stable humus like character, is carried in suspension and can easily be removed by sedimentation. For the purpose of removing the flocculent matter from sprinkling filter effluents, a sediment basin, through which the effluent must pass, is generally one of the parts of a sprinkling filter plant. The suspended solids removed from sprinkling effluents by sedimentation are sometimes in themselves putrescible, but usually contain such a small proportion of unstable matter that they can be discharged into a watercourse without producing a nuisance. The effluents look worse and keep better than would be expected from their chemical composition.

The value of sprinkling filters lies in the fact that they can purify to a nonpurificative stage a much larger quantity of sewage than can be purified to a like degree by any other process. The trickling effluents, however, are not as pure as those from intermittent sand filters, although they are in general better than those yielded by contact beds or overworked sewage farms. At the present time, sprinkling filters are used as a secondary or final process, following straining and septic action. It seems quite possible, however, that in some cases, for instance where sewage has flowed a great distance in sewers, and is there

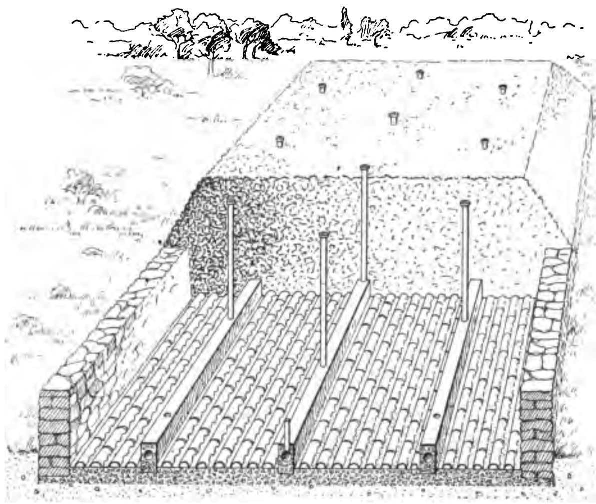


Fig. 39

subjected to septic action, that the crude sewage might be applied to sprinkling filters, after first passing through a screening chamber to remove the coarser solids.

Examples of Sprinkling Filters

A sprinkling filter consists of a bed of coarse material, like crushed stone, coal or slag, placed on a suitably graded impervious flooring, that slopes toward a drain, so that the effluent from the sewage sprayed onto the bed can be collected and discharged at a convenient point. A sprinkling filter does not require an impervious or watertight basin, nor does it need to be enclosed by walls. On the contrary, if enclosed in a basin, or confined by walls, the masonry should be laid up without mortar, and with wide open joints so that the air can freely circulate through the interstices of the filtering material.

Great diversity of design is exhibited in the construction of sprinkling filters. The material of some filter beds is left unconfined so that the material around the edges ultimately settles down to the angle of repose. In other beds, the material is partly confined by low stone or brick walls, while in other filter beds the material is confined by hand-piling the larger filtering pieces around the edges, so that the bed will retain its shape without settling to the angle of repose.

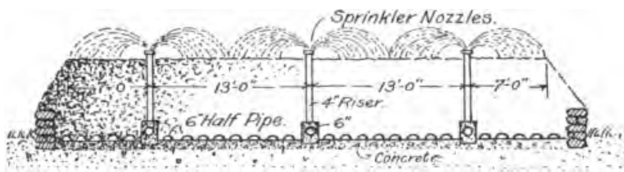


Fig. 40

Sewage is applied to sprinkling filters in a variety of ways. In some filters, the sewage is sprayed from perforated pipes, laid on the surface of the filter bed. On others, sprinkling nozzles are used, which may be attached to stand-pipes rising from distributing mains buried in the bed, or may be connected to overhead pipes, so they will spray downward. Other filter beds have revolving perforated arms which spray the sewage; while on still others, the sewage is distributed from traveling water wheels which are actuated by the sewage. An example of a rectangular sprinkling filter confined by low walls is shown in Fig. 39. The underdrain tiles rest on a floor of concrete which slopes toward a collector that may be located either at one end of the filter or at the center. The distributing mains are made of cement, and are built integral with the floor on which they rest; from the distributing mains at suitable distances rise cast iron pipes which are surmounted by spraying nozzles, through which the sewage is sprayed onto the surface of the bed. The sprinkling filter shown in this illustration is built above ground, which is a common practice.

When necessary, however, they may be built below the level of the ground, but in that case sufficient space must be allowed at all sides of the bed to permit of aëration. Sometimes walls enclosing sprinkling filters,

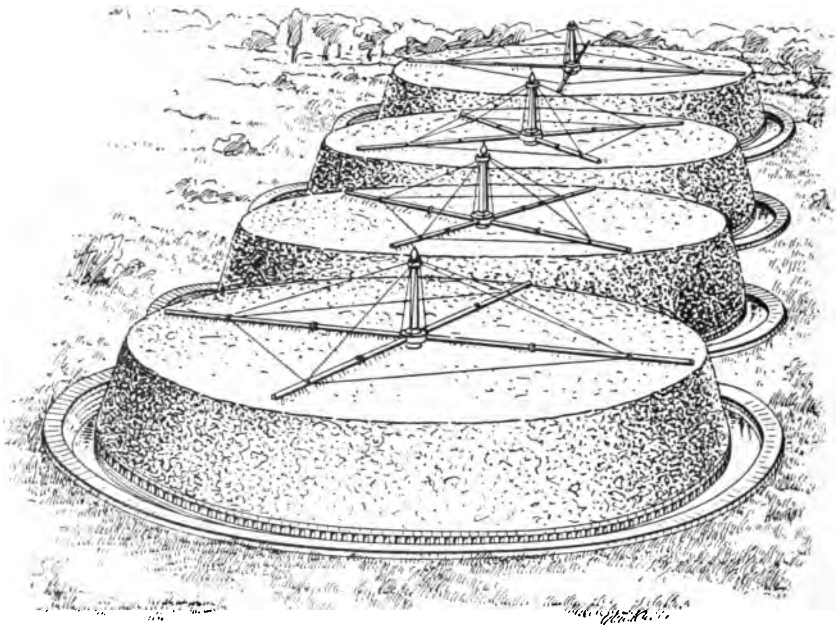


Fig. 41

particularly the dividing walls between filters, are built solid, and tiles built into the walls to admit air.

A sectional illustration, giving the dimensions and relation of the various parts of a sprinkling filter, is shown in Fig. 40. This illustration is only suggestive, however, as the dimensions and arrangement can be changed to suit various requirements as they may arise.

A battery of sprinkling filters fitted with revolving arms is shown in Fig. 41. These beds are made circular so the spray will reach all parts of the filtering material. The filtering material is hand-packed so the pile will hold its shape, and the entire bed rests on a floor of bricks underdrained by other bricks standing on edge, and radiating from the center of the bed to the circumference where the effluent discharges freely into an open gutter. The gutter is graded so that the effluent can be collected and discharged from the lowest point.

The floor of a round filter bed is generally made to slope from the center towards the outer perimeter, and is covered with split-tile, open brickwork, or some other kind of drain which will facilitate drainage and at the same time permit a free circulation of air.

SPRINKLING FILTER DETAILS

Materials for Sprinkling Filter Beds—Considerable data have been collected regarding the materials most suitable for sprinkling filters, and as a general rule, derived from the data, it may be said that any hard, durable material will serve well for this purpose. The chief requisite is that the material be of a stable character which will not soften or disintegrate under the action of water. Anthracite coal has been found particularly suitable for this purpose, but on account of the cost of coal, it is not extensively used. Coke, slag and furnace cinders, each has been successfully used, and in many localities are preferable to other material on account of their low cost. Granite, trap rock, flints, gravel, hard clinker, slate, blast furnace slag, boiler slag and broken bricks, have all been successfully employed for sprinkling filter beds. Smooth material is not so good as rough material when a filter bed is first put into service, but the difference soon disappears as the filters grow old, after which time no difference in efficiency can be detected. As a rule, the material to use for a sprinkling filter bed will depend on the availability of the several materials in the locality where the plant is to be constructed. There is hardly a part of the civilized world where some one of the suitable materials is not at hand,

and can be had at low cost; and the cost of the material will generally determine the choice.

Size of Material for Sprinkling Filter Beds—The size of material for sprinkling filter beds may vary considerably within certain limits, but the various sizes of filtering material within any one bed should be proportioned to one another. For instance, the size of materials may vary from 1/8 inch to 2 1/2 inches but those extreme sizes should never be placed in the same bed. Ordinarily, the extreme proportion or ratio should be as one to four. That is, in a filter bed in which the smallest size material is 1/8-inch, the largest size should not be over 1/2-inch, while in a filter bed in which the smallest size of material is 1/2-inch, the largest should not exceed 2 inches. The usual sizes of filtering material that go together are 1/8-inch to 1/2-inch; 1/4-inch to 3/4-inch; 3/8-inch to 1-inch; 1/2-inch to 1 1/2-inches; 3/4-inch to 2-inches; 1-inch to 2 1/2-inches.

The sizes of stones for filtering material are determined by the size mesh of screen they will pass through; for instance, a 1/2-inch filtering material is a material which will pass through a screen of 1/2-inch mesh, and will be held back by a screen of 3/8-inch mesh. When two sizes of stones are specified for a filtering material, pieces of just those two sizes are not intended, but any size falling between those two are included. For instance, material from 1/8-inch to 1/2-inch means all the material which will pass through a 1/2-inch screen and will be held back by a 1/16-inch screen. It may safely be assumed, that the smaller the medium the more complete will be the purification obtained for beds of equal depth, but when the beds are proportioned in depth to the size of material there is but little difference between the effluent obtained from beds with different size materials; and when treating a like sewage but little better result can be obtained by the use of 1/8-inch to 1/2-inch material than can be obtained by 1/2-inch to 1 1/2-inch material. There are other conditions, however, which determine the size of material most suitable to use. For instance, if the filtering material is softer than is desired, but is the best material available at a reasonable cost, the better practice no doubt would be to use large sizes to prevent the bed clogging from the disintegrating portion.

The power of coarse sprinkling filters to unload stored material in a condition so that it can be easily disposed of on land without creating a nuisance is of much practical value when treating a strong sewage, and is a feature that will recommend coarse filters to most engineers.

Large material is likewise used when constructing a filter to care for an exceptionally large quantity of sewage, as large materials permit a freer vertical circulation of air. The larger sizes of material are less liable to clog than would the smaller sizes, and the larger sizes will permit better aëration of the bed than would fine filtering materials. Fine filtering material, on the other hand, is preferable when sewage is to be applied at a moderate rate and a better effluent is desired.

Depth of Sprinkling Filter Beds—The depth of filtering materials for a sprinkling filter depends somewhat on the degree of purification desired. It might be said that the purification effected is proportioned to the depth of the bed, but not in direct proportion, nor is the additional purification sufficient to warrant extending a bed to a very great depth. The minimum depth for a sprinkling filter bed is perhaps four feet, but a greater depth is desirable on account of the danger of raw sewage passing through channels in shallow filters due to irregular packing of the material. On the other hand, a sprinkling filter bed seldom need be made deeper than ten feet. The purification effected in any additional depth would hardly be sufficient to warrant the expense. As a basis for estimating the depth of the filter beds it might be assumed that five feet will be sufficient when the effluent discharges into tide water, or into the lower reaches of a river below the intake to a waterworks pumping station; eight feet where the effluent discharges into a river having a dry weather flow of over 45 times the volume of sewage, and ten feet where the effluent discharges into smaller streams.

Period of Flow Through Sprinkling Filters—The period of flow through sprinkling filter beds varies from two or three minutes to thirty minutes. In beds made of large materials, seldom more than three minutes are consumed in passing through the bed, while in materials ranging in size from ⅛-inch to ½-inch, thirty minutes is not an unusual period.

DISTRIBUTORS FOR SPRINKLING FILTERS

Fixed Sprinklers—The ideal method for the distribution of sewage on a sprinkling filter would be to discharge it continuously in a fine even spray over the entire surface of the filter. The sewage being finely divided in the form of spray would have every particle exposed to the atmosphere and would enter the voids of the filter bed well charged with air, besides carrying along more by its momentum.

The ideal, however, is never attained in practice. With revolving sprinklers and traveling distributors the application is intermittent instead of continuous; and with fixed sprinklers, owing to certain limitations, the entire bed can never be sprayed. When fixed sprinklers are used, each sprinkler nozzle occupies the center of an imaginary square on the surface of the filter bed, and, as the spray from the sprinkler's nozzle falls in a circle, the four corners of the square will remain unwetted. The area of a circle inscribed in a square being only .7854 of the area of the square, it follows, theoretically, that 78 per cent. of the surface is all that can be covered by a fixed sprinkler nozzle. As a matter of fact, however, the percentage is somewhat increased by staggering the sprinkler nozzle, so that instead of having a square surface to cover, each sprinkler is set in the center of an irregular surface, bounded by the spray from the other sprinkler heads, so that actually about 92 per cent. of the alloted surface is covered when the sprinklers are spaced at the correct distance for the heads they are operated under.

Of the surface covered by spray from fixed sprinkers, the distribution is not uniform with all types of sprinkler heads, but, on the contrary, varies at different distances from the sprinkler nozzle; so that if the surface covered by the spray be divided into a series of concentric circles, some of the circles will receive more spray than others. This is not true of all sprinkler heads. On the contrary, the new Salford nozzle, also the Birmingham nozzle, effect an almost perfect distribution of sewage over the surface wetted. The openings to these nozzles are so small, however, that they are liable to cause trouble from clogging when coarse sewage is used. The openings to the Salford nozzle are only $\frac{5}{32}$-inch in diameter, while that of the Birmingham nozzle is an annular ring only $\frac{5}{64}$-inch wide. In the American types of sprinkler heads, like the Columbus, in which the openings are made larger, the distribution is much less uniform. This objection may be overcome, however, by varying the head on the nozzle so that the points of maximum or excessive wetting will be constantly changing, thus making the distribution over the entire surface almost uniform. The head may be varied on the sprinkler nozzles either by dosing from duplicate tanks, which are filled and discharged alternately, or by means of automatic siphons. It is found by experiment that with varying heads the distribution is better when the tanks are not too long in emptying, since with the low heads, that is, anything less than three feet, the distribution is not so good as with the greater heads.

Wind is another factor which interferes with the uniform distribution from sprinkling filters. This feature, however, can be overcome or the effect minimized only by selecting a site where the prevailing winds are not strong or, in extreme cases, by constructing a wind-break.

The method of applying sewage to sprinkling filters varies in different countries, preference being given in America to distribution by fixed sprinklers, while in Great Britain revolving sprinklers and traveling distributors seem to have the preference.

A system of fixed sprinklers for a rectangular filter bed is shown, both in plan and in section, in Fig. 42. In a system of this kind, each distributing main is controlled by

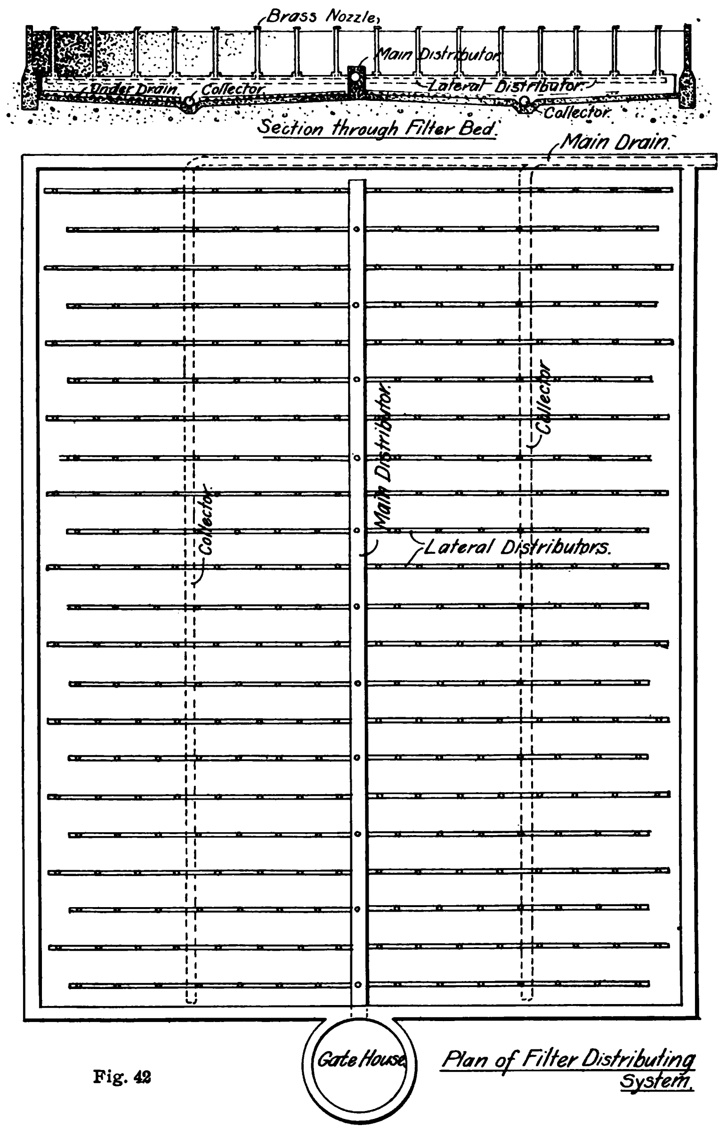


Fig. 42

a valve located in a gate house, so that sewage can be shut off from that part of the filter supplied by the main.

The distributing mains may be made of cement concrete, or may be of iron. In the latter case, cast-iron pipe should be used, as wrought-iron and steel pipe have comparatively short lives when used in such manner. Generally, the lateral distributors are made of earthernware pipe embedded in concrete. It is a good practice to provide means for flushing the distributing mains to remove the deposits which accumulate therein, due to the depository velocities maintained to reduce the head that otherwise would be lost by friction.

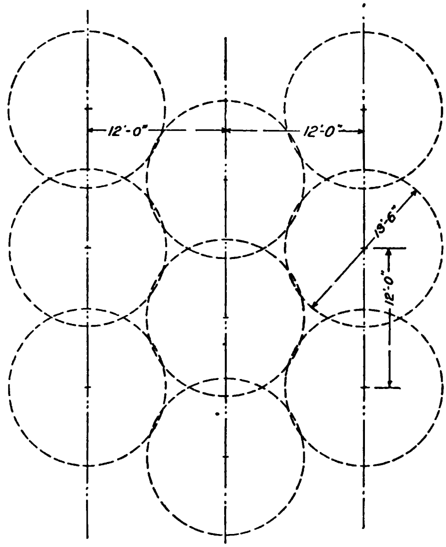


Fig. 43

Instead of burying the main distributors near the bottom of the filter bed, in some plants the mains are laid on top of the bed and the sprinklers fitted, hand tight, into the holes of bosses made specially for this purpose. In other plants the mains are buried about one foot from the surface, while in others, which are more economically built, and designed to operate under a lower head, perforated salt-glazed pipe distributors are laid along the surface of the beds, about six feet apart; the tile pipes are from 3 to 4 inches in diameter and each length is perforated with two ¼ or ⅜-inch holes, through which the sewage is sprayed onto the beds.

It will be noticed that the vertical branches from the lateral distributors to the nozzles are not tapped in a direct line across from one another, but, on the contrary, are staggered. This is done to obtain a better distribution of the sewage than would be effected if the outlets were all in line. A reference to Fig. 43 will show that when the sprinklers are staggered, there is but little of the surface which receives a double dose of sewage, and very little surface to which sewage is not applied; so little, in fact, that, owing to splashing, action of the wind, and lateral movement of the sewage through the filtering material, within a few inches below the surface of the bed there would doubtless be a fairly uniform flow of sewage over the filtering material. In the case of Fig. 44, however, the result is different. Here the sprinklers are all in line, and, spaced equal distances apart and with the same diameter of spray as used when the pipes are staggered, there would be parts, *a*, *a*, of the filter which would not be sprayed, while other parts, *b*, *b*, would receive a double dose. If a less diameter of spray be adopted in this case, while little or no surface would receive a double dose of sewage, the parts actually sprayed would be reduced to such an extent that a large part of the filter surface would be useless.

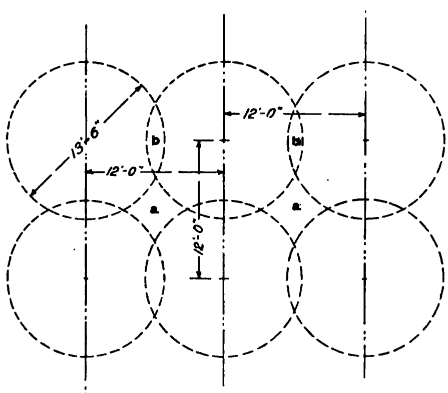


Fig. 44

The distance apart that sprinklers can be spaced depends entirely upon the available head of sewage and the volume of discharge. The lowest workable head for the average sprinkler nozzle is 2 feet 9 inches, and the most effective head about 6 feet. Nozzles differ so, in many respects, that in each case the type to be used should be tested to find out the diameter of spray and volume discharged for each foot of head. Ordinarily, for each foot of head available, the average jet will cover a surface approximately 2 feet in diameter, consequently, for rough calculation, it may be assumed that, with a 4½-foot head, the sprinklers may be spaced 9 feet apart; with a 5-foot head, 10 feet apart; with a 5½-foot head, 11 feet apart, and with a 6-foot head, 12 feet apart, provided the quantity of sewage discharged is proportioned to the area covered. There is a certain intimate relation between the quantity of sewage discharged from a sprinkler nozzle, the head required to produce the discharge and the area of filter surface covered which must not be overlooked. For instance, in the American types of sprinkler nozzles, the openings are made large to prevent stoppage. In the Columbus sprinkler nozzle, operating under a 4.3-foot head, the discharge is 10.9 gallons per minute. At this rate of discharge, 131 sprinkler nozzles are required per acre to equal a rate of filtration of 2,000,000 gallons per 24 hours. At this rate of filtration the sprinklers would have to be spaced 19 feet apart, but the nozzle operating under a 4.3-foot head would cover an area only 12 feet in diameter, so that while the filter bed would be treating sewage at the rate of 2,000,000 gallons per 24 hours, the sewage would be treated on only 14,816 square feet of surface out of 43,560 square feet in an acre, or on about one-third of the surface, which would equal a rate of about 6,000,000 gallons per 24 hours. To overcome this, on some filters a head of 7½ feet is maintained on the sprinkler nozzles, or a variable head alternating from 7½ feet, with a discharge of 13.5 gallons per minute, to a head of 4.3, with a discharge of 10.9 gallons per minute. At a rate of discharge of 13.5 gallons per minute, the nozzles can be spaced 16.2 feet apart and the surface between them well wetted with sewage. Spacing the nozzles 16.2 feet apart would require 180 nozzles to an acre, and at a flow of 13.5 gallons per nozzle, would be equal to a filtration capacity of about 3,500,000 gallons per acre per day. This quantity, however, would be sprinkled on about 92 per cent. of the filter area, which would increase the actual rate of filtration through the surface wetted to approximately 4,000,000 gallons per acre per 24 hours. Where plants are designed for this rate of filtration, the nozzles are in operation only one-half the time, so that, while the rate of filtration while in operation is 4,000,000 gallons per acre per day, the actual quantity filtered is only about 2,000,000 gallons per acre per day. To secure an actual rate of filtration of 2,000,000 gallons per acre per 24 hours, actual operation, while spraying continuously, the nozzles can be operated under a variable head from 4 feet to 2 feet. This gives a discharge of approximately 4 gallons per minute, distributed over an area of 10½ feet in diameter, and permits spacing the nozzles about 11 feet apart, which is equivalent to about 390 nozzles per acre. Sixteen feet two inches is the greatest distance that sprinklers are spaced in the present state of sewage purification practice.

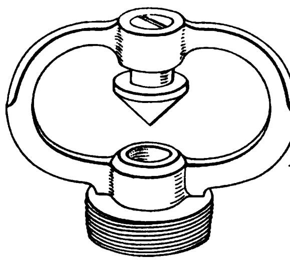


Fig. 45

Sprinkler Nozzles—A sprinkler nozzle which is both simple and effective, and which will work under a low head, is shown in Fig. 45. The nozzle, which is known as the Columbus, consists of a single orifice, $\frac{9}{16}$-inch in diameter, above which an inverted cone is held by two thin arms, the axis of the cone coinciding with the axis of the orifice. The jet upon leaving the orifice impinges against the cone and is transformed into a thin sheet, which spreads out radially and then breaks into a mass of drops, which fall on an area included within a circle of a diameter depending on the head. The principal objection to this type of nozzle lies in the fact that the two arms which hold the inverted cone in place,

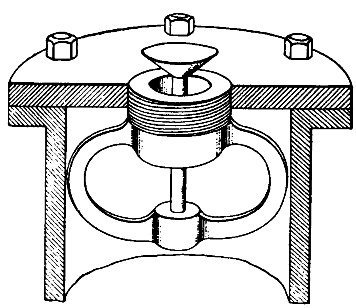


Fig. 46

theoretically, protect a certain part of the bed from being sprayed with sewage; as a matter of fact, however, the blades or arms are so thin at their edges that the sheet of water, after having been separated, closes up again after passing the arms, so that sewage sprinkled from such a nozzle will cover the entire surface included within its range. However, a modified design of this type of sprinkler head is now made which does away with that objection. This sprinkler head is shown in Fig. 46. It consists of an inverted frustum of a cone, carried by a rod passing through the center of a $\frac{15}{16}$-inch nozzle orifice. The cone slopes 45 degrees, is $1\frac{1}{4}$ inches in diameter across the top, and is

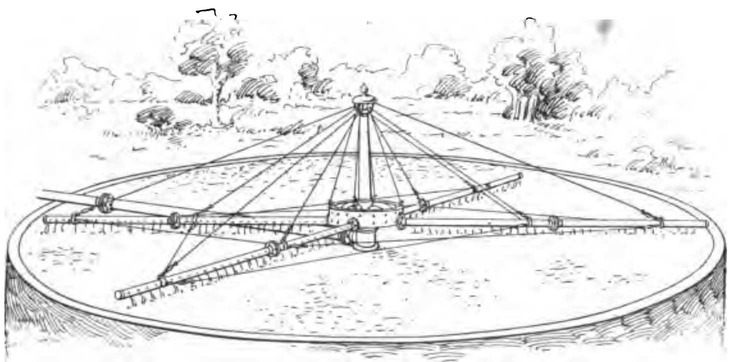


Fig. 47

carried by a $\frac{5}{16}$-inch rod. The junction of the rod and the cone is $\frac{3}{16}$-inch above the plane of the orifice. To operate satisfactorily the rod must be kept exactly in the center of the orifice.

Proportioning Distributing Systems—Larger pipes are allowed in designing the distributing system for a sprinkling filter than would be required under different conditions to conduct equal quantities of water or sewage. The extra capacity of pipes is allowed to reduce to the minimum the loss of head from friction, compensate for the reduction in size due to tuberculation and sedimentation, and to provide a large volume of sewage to maintain a high temperature, thus preventing the sewage from freezing. In most of the large sprinkling filters now in use, 3-inch cast-iron pipes are used as risers to supply the sprinkler heads, which discharge sewage at the rate of from 10 gallons per minute, when spraying at the minimum rate, up to 13.5 gallons per minute, when operating at the maximum rate.

Revolving Sprinklers—Revolving sprinklers are used principally in Great Britain. They consist simply of radial perforated tubes, which are caused to revolve around a center axis by means of the sewage flowing through the pipes and acting on the vanes of a water wheel. The construction of a revolving sprinkler is shown in perspective in Fig. 47, and the manner in which it operates is illustrated in Fig. 48. The weight of the apparatus is supported by the upright, *a*, and the movable part revolves on ball bearings, *b*, located in the head casing. Sewage is conducted into the distributor through the pipe, *c*, and issues from the central pillar, *d*, through fixed pipes, *e*, *e*. As the sewage flows from the mouths of these pipes, it impinges against the blades, *f*, *f*. which are attached to the trough, and gives the arms an impulse to the left. The sewage then falls to the bottom of the trough, passes into

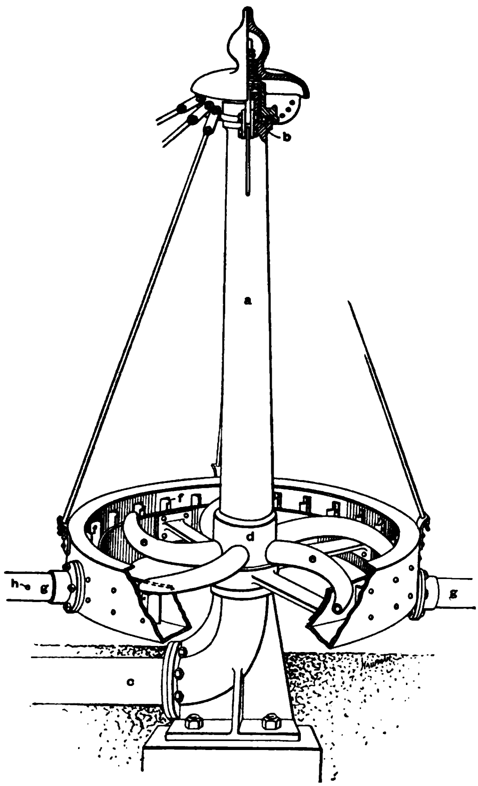


Fig. 48

the four radial arms, *g*, *g*, and, issuing from perforations, *h*, in the right side of the arms, gives the sprinkler a further impulse, the combined efforts of which cause the sprinkler to revolve. These sprinklers are made with two, four and five arms, and are designed to distribute sewage at the rate of 200 gallons per square yard per day of 24 hours' continuous flow. Sprinklers of two arms are used for beds of 60 feet diameter and smaller, while four and five-arm sprinklers are used for beds of larger diameters. Revolving sprinklers have been successfully used in England on beds 130 feet in diameter.

Revolving sprinklers require considerable attention to

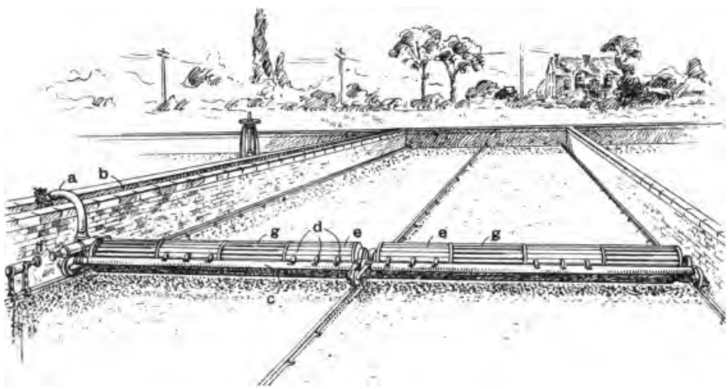


Fig. 49

keep them in working order, daily cleaning with brushes sometimes being necessary to prevent the holes from being clogged.

Automatic Traveling Distributor—A sewage distributor for a rectangular bed, which distributes sewage by splashing or emptying it from the buckets of traveling water wheels, is shown in Fig. 49. This distributor receives sewage through pipe *a* from the trough *b*, which extends the entire length of the filter bed. From the pipe *a* the sewage flows into pipe *c*, or into a similar pipe on the opposite side of the distributor, depending on which way the apparatus is traveling. Assuming that the apparatus is approaching, the flow of sewage would be through pipe *c* and spouts *d*, *d*, into the buckets of each alternate wheel *e*, *e*. The weight of sewage in these buckets causes the wheels to revolve and the distributor to travel along. When the distributor reaches a certain point, the valve stem *f* strikes a tripping device, which shuts off the flow of sewage from pipe *c*, and shunts it into the similar pipe on the opposite side. From this pipe, the sewage overflows through spouts into the buckets of wheels *g*, *g*, causing them to revolve in the opposite direction, carrying the entire apparatus along. It will be observed that the appli-

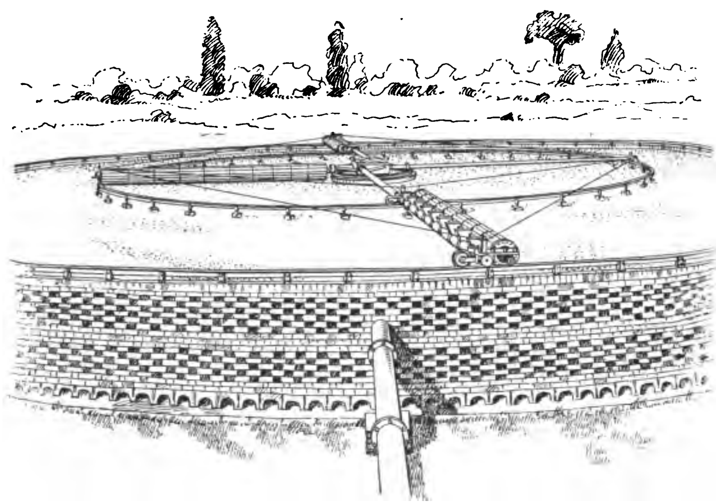


Fig. 50

cation of sewage by means of this apparatus is intermittent instead of continuous, and that sewage is applied to only half the width of the bed when the apparatus is moving along.

Automatic Revolving Distributors—The splash system of distribution is likewise adapted to circular beds, as shown in Fig. 50. The only difference is that instead of a reciprocating movement, the water wheels travel continuously in a circle. In this manner, while the application of sewage is intermittent, the entire width of the filter bed is wetted while the water wheel is traveling over it.

Capacity of Sprinkling Filters—The rate at which sprinkling filters can be operated seems to lie somewhere between one million gallons per acre per day and three million gallons per acre per day. Perhaps two million gallons per acre per day is a fair average when the filters are working continuously, or at the rate of four million gallons per twenty-four hours when resting half the time. The rate of filtration that can successfully be used depends considerably on the strength of the applied sewage. When treating an ordinary American sewage containing much street refuse, a rate of two million gallons per acre per day can be safely adopted. When treating sewage from a city having a separate system of sewers, or when treating septic effluent from country institutions, the septic effluent can be applied at the rate of two and one-half million gallons per acre per day, which is equal to a rate of about 500 gallons per square yard per day. Allowing a per capita consumption of water of 125 gallons per day, one acre of filter surface at that rate would care for the sewage from a city of 20,000 population.

Efficiency of Sprinkling Filters—Sprinkling filters are not a substitute for intermittent sand filters, which remove practically all the matter in suspension in the sewage, but are simply devices for the quick oxidation of the putrefying matters in sewage, while allowing the larger body of stable or slowly decomposing matters to pass along with the effluent. The purification effected by sprinkling filters is good, and would be considered better in general than that obtained by the double contact process. The effluent from a sprinkling filter is likewise better than those yielded by single contact beds and sewage farms which are overworked, but is not so good as that produced by intermittent filtration. The effluent from sprinkling filters contain as a rule a certain amount of flocculent organic matter which mars its appearance, but this matter has been more or less oxidized and is of a stable, humus-like character. The suspended solids in sprinkling effluents can easily be removed by a short sedimentation. It may safely be assumed that sprinkling filters operated at proper rates for the strength of sewage applied will produce effluents containing nitrates averaging two parts per 100,000. The effluent from the filters will contain approximately 50 per cent. of the organic matter in the applied sewage, but this organic matter will be freed from most of its easily putrescible bodies, so that the residue of organic matter will be stable. There is a visible difference between the effluents from coarse-grained and fine-grained sprinkling filters. The effluents from coarse-grained filters are not clear, but contain large amounts of suspended flocculent matter, while the effluents from fine-grained filters may be clear. The effluents from sprinkling filters, either coarse-grained or fine-grained, are non-putrescible, but bacteriologically, they are not much better than the crude sewage, and are not good enough to be turned into streams which are used without filtration for water supply, or into bays or tidal estuaries where oyster beds are located.

Settling Basins—Settling basins are usually provided to remove from sprinkling-filter effluents the suspended organic matter which is of a fairly coarse nature. These basins may be made in any desired shape, should have a capacity of from two to three hours' flow, and should have a depth of about ten feet. The bottom of the basin should slope toward a sludge outlet to facilitate the removal of sludge. Effluent should flow through the settling chambers at a very low velocity, probably at a rate not greater than two inches per minute, to allow time for the sludge to settle. About one cubic yard of sludge will accumulate in settling basins for each 1,000,000 gallons of effluent passing through. Shallow tanks are sometimes successfully used, and the present tendency of practice is to interpose a screen composed of two wire nettings about 12 inches apart and filled with coke, between the inlet and outlet to the tank.

Odors from Sprinkling Filters—Where sprinkling filters are properly designed, constructed and operated, no objectionable odors should be noticeable beyond one-fourth mile from large plants, and usually not beyond a few hundred feet. The odors from small plants will be noticeable at correspondingly less distances.

Permanency of Sprinkling Filter Beds—The filtering material in sprinkling filter beds will probably require removing, cleaning and replacing once in from ten to fifteen years. Occasionally, however, the underdrains may require flushing to wash out deposits of organic matter, and deposits might sometimes accumulate in the distributing mains or branches.

Effect of Temperature on Sprinkling Filters—No data outside of experimental data, are obtainable showing the effect of severe winter weather on sprinkling filters. In the absence of definite knowledge it may safely be assumed that sprinkling filters cannot successfully be used in localities having a mean winter temperature below twenty degrees Fahrenheit.

CONTACT BEDS

Principles of the Contact Bed—If sewage be discharged into a watertight tank, which is filled with broken stone, coal, glass, coke or other material; is allowed to stand there in contact with the filling material for a certain period of time, and the effluent is then withdrawn so the interstices of the material can fill with air, it will be found that a material purification has been effected in the sewage, from which a considerable portion of the organic matter has disappeared, while the residue, which passes off with the effluent, has been freed from most of its easily putrefied compounds, so that it is of sufficient stable quality to create no nuisance.

The filling material in a contact bed is covered with a jelly-like film, which forms a habitat for the reducing and nitrifying micro-organisms, and the filling material helps to entangle organic matter in these films by the attractive power the particles exert on the suspended matter in the sewage. The length of time that the sewage is in contact with the filling material is an important consideration. If the period be too short, sufficient purification will not take place, while if the period be prolonged beyond the proper stage, putrefaction will begin, producing a dark, disagreeable-smelling effluent, which is difficult to purify.

The process which takes place in a contact bed is evidently a very complex one, where either aërobic and anaërobic processes take place alternately or they are carried on simultaneously side by side. During the process the nitrates formed while the bed is empty are partly or wholly consumed during the period of contact, and for this reason the nitrates found in the final effluent from a contact bed are not a true measure of the purification effected, since under favorable conditions the nitrates formed from half the nitrogen would be used up in decomposing the other half.

To be successfully operated, a contact bed should be allowed sufficiently long and frequent periods of rest, and the sewage applied to the beds should be of uniform character and free as possible from suspended matter. On this account, some form of preliminary treatment is advisable before applying sewage to contact beds, particularly when only single contact is provided for. Septic tanks, or sediment tanks, are the preliminary treatment commonly resorted to in connection with contact beds. The treatment of septic effluent instead of crude sewage greatly prolongs the life of a contact bed, while at the same time the loss of water capacity is less, and consequently the capacity of the contact bed greater, when treating septic effluent.

Example of a Single Contact Bed—A single contact bed is shown in perspective in Fig. 51. The tank enclosing a contact bed must be watertight, and, for this reason, is generally made of masonry or concrete. Sometimes, however, puddled clay reservoirs are used for this purpose, while in other installations the bottom is of clay and the walls of concrete. In many respects a contact bed resembles an intermittent filter. The system of underdrains is the same, with the exception that the outlet is controlled by a sluice gate, *a*, to cut off the flow and hold the sewage in contact for the full period. The filling material for a contact bed is much coarser than the sand in an intermittent filter bed, but is of about the same depth. Distributors for contact beds are similar to those used to distribute sewage on intermittent sand filters, although, instead of wooden or concrete sluices, trenches may be made of cinders, which will allow the liquid portion of the influent to seep through and at the same time act as a strainer to hold back the coarser particles of matter. When the cinder sluices are used they are shaped roughly like wooden sluices and likewise have branches to conduct sewage to various parts of the bed. The surface of the filling material is not provided with

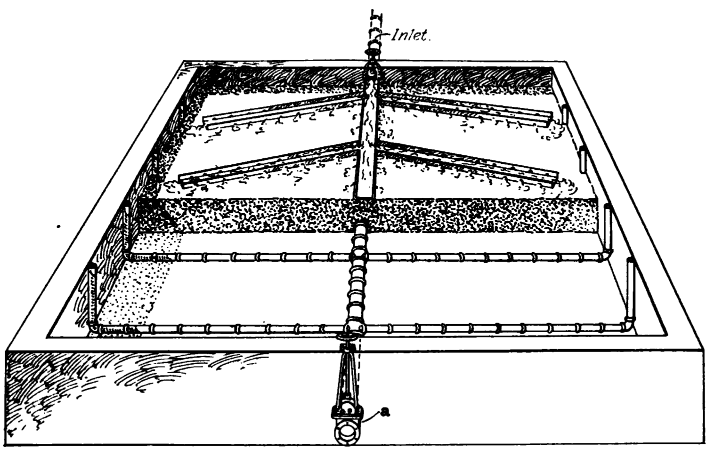


Fig. 51

pavement where water leaves the troughs, however, as the material of the contact bed is too coarse to be easily disturbed, and even if it were easily disturbed it is not vitally important to keep the surface of the bed level.

Material for Contact Beds—The materials suitable for contact beds are so numerous that fortunately some suitable material can be found in every locality. It is advisable when selecting a material to choose one which while it might not be the best so far as efficiency is concerned, will at all events be permanent and thus keep down the operating expenses of the plant. Coke and coal seem particularly favorable for contact beds, but when cost is considered it might be found advisable to use some poorer material that can be had at less cost, depending if necessary on double contact beds to produce equal results.

The presence of iron in a contact bed seems to exert a favorable influence, which might account for the results obtained by use of broken bricks and other burnt clay products. Rough materials, as a rule, give better results than smooth materials.

It would seem that coke, coal, slag, bricks, gravel and broken stone rank in efficiency in about the order named. Broken stone filters are not particularly efficient. However, on account of the availability and the permanency of stone, contact beds of this material are extensively used. Burnt clay in the form of bricks, broken tile, etc., give very good results, but unless the bricks are hard and the tile vitrified, they break down badly, clogging the bed, decreasing its capacity and necessitating cleaning.

It might be stated that, as a rule, any hard, durable material which can be broken or crushed into particles of the required size may successfully be used as a filling material for contact beds. Various kinds of stone, such as traprock, granite, slate and limestone, also glass, have been used, and under proper conditions give satisfactory results. Whatever material is used, however, it must be screened and freed from dust and fine particles before being deposited in the contact basin.

Size of Material for Contact Beds—The size of material for a contact bed is of more importance than the kind of material used. As might be expected, the smaller the material of a contact bed the greater the degree of purification attained. The reason is, that the finer the material of a contact bed, the greater is the active surface occupied by the micro-organisms, and with which the organic matter in the sewage comes in contact. Greater purification would, therefore, follow as a natural sequence. This higher degree of purification, however, will be accompanied by greater loss of capacity, particularly if the fine material be used in a primary contact bed. For this reason, where double contact beds are constructed, the primary bed is filled with a coarse material of from about ⅓ inch to 1⅛ inches in size, and the secondary beds are filled with material from about ⅛ inch to ⅓ inch in size. The material in the primary contact bed is supposed to be large enough to admit into its voids the particles of suspended matter carried by the sewage, yet fine enough to prevent their being washed out onto the secondary beds. The fine-grained secondary bed, on the other hand, will admit into its interstices any suspended matter discharged from the primary bed and will produce an effluent free from suspended matter.

Depth of Material for Contact Beds—Contact beds are usually filled to a depth of from 4 to 5 feet with the contact material. Less depth than 4 feet could be used, but in that case the area covered by the beds would have to be increased to make up for the decreased capacity caused by the shallowness of the beds. Greater depths than 5 feet are not advantageous and would require walls of too great thickness to confine the head of water within. If the filling material be under ⅛ inch in size, the depth of bed should not exceed 3 feet.

Liquid Capacity of Contact Beds—Contact beds, when first put into service, have a liquid capacity of about 50 per cent. of the total cubical contents of the bed. The original liquid capacity, however, is soon reduced to about 33 per cent. by the settling together of the material; growth of organisms; breaking down of material; impaired drainage and insoluble matter entering the beds. With progressive use, the capacity decreases until a point is reached where the filtering material must be removed from the beds and cleaned or renewed. This is one of the most objectionable and costly features in the operation of a contact bed.

The original capacity of contact beds can be restored to a considerable extent by allowing the beds to rest empty for several weeks. This, of course, will affect only the organic matter and growth, and cannot affect the accumulated mineral matter. In practice it is generally assumed that after being in service awhile, the liquid capacity of a contact bed treating septic effluent will be 33 per cent. of its cubical capacity, and when treating crude sewage 25 per cent. of its cubical capacity. The liquid capacity of a contact bed is determined by measuring the effluent drawn off when a bed is emptied.

Operation of Contact Beds—Contact beds are operated in a series of cycles. Usually the cycle consists of: One hour filling, two hours resting full "in contact," one hour for draining and four hours resting empty—a cycle of eight hours, after which it is again filled. When carefully managed and operated, double contact beds can receive three fillings in twenty-four hours, while for single contact beds, two fillings give the best results. Two fillings a day give better results than one filling. It is probable that one filling does not maintain the bacteria in their maximum effectiveness. The distribution of fillings at regular intervals over the twenty-four hours is not necessary to the successful working of a plant. When a bed is first started the purification is very slight, but the bacteria soon multiply enormously, clinging in colonies to the surfaces of the filling material. In about two or three weeks the bed becomes ripe, after which it will maintain its efficiency indefinitely if properly handled. A sewage which is slightly alkaline, the normal condition of ordinary domestic sewage, is the most favorable for bacterial activity in a contact bed. Very alkaline effluents, however, such as those produced by the use of lime in excessive quantities, are very liable to putrefy instead of being purified by oxidizing organisms. That is probably why chemical treatment with lime interferes with the process in a contact bed. The period of contact is an important consideration in the operation of a contact bed. If sufficient time is not allowed, the purification will be incomplete, while, if the period of contact be carried too far, the aëration of the bed and consequent recuperation are unfavorably affected. Two hours have been found to be about the best period of contact; the effluent is then allowed to escape, while the organic matter held in the bed is attacked by the micro-organisms under the most favorable conditions of dampness and air supply.

Capacity of Contact Beds—The rate at which contact beds can be operated depends on the strength of the applied sewage and the condition in which it is delivered to the beds. A greater quantity of weak domestic sewage, which has been subjected to septic action, can be treated than of strong sewage, which is only strained or subjected to a short period of sedimentation. The number of fillings which a contact bed will stand also has a marked effect on the quantity of sewage that can be purified. Usually a contact bed can treat more septic effluent than it can crude sewage, but at Hamburg the converse was true. There the contact beds could handle six doses of crude sewage a day, while only two doses of septic effluent could be applied, a third dose producing a dark, malodorous effluent.

In the present state of sewage purification practice by the contact method, three fillings each day is about the maximum rate for primary beds, while two fillings per day will give the best average results. When a double contact system is used, the secondary beds usually occupy about one-half the area of the primary beds and are operated at double rates. The quantity of sewage which can be purified per acre of surface under average conditions varies from about .8 million gallons per day to 1.2 million gallons per day, with perhaps a mean of 1 million gallons per acre per day, which may safely be assumed as the average capacity of a contact bed. Experimental contact beds operated in Boston for the study of the best treatment for Boston sewage were operated most satisfactorily with three fillings a day, giving a single contact rate for 6-foot beds, with fine stone filling, of 1.2 million gallons per acre per day; with coarse stone, 1.4 million gallons per day; 1.8 million gallons with 2-inch coke and with 2-inch brick; it was found, however, that with any material other than the ½-inch stone a second contact would be necessary, reducing the rate on the double system as a whole to between .6 and 1 million gallons. Table XII, compiled by Winslow and Phelps,* shows the rates which have recently been obtained in actual operation or in experiment on a practical scale.

Efficiency of Contact Beds—Single contact rarely yields a stable effluent without the beds clogging to such an extent that they necessitate the removal of the material several times a year for cleaning.

TABLE XII—CONTACT FILTER RATES

SINGLE CONTACT

Place	Depth Feet	Rate Million Gallons per Acre per Day
Manchester	3.3	.6
Birmingham	4.5	.6
Croydon	3.7	.8
Exeter	5	1
Sutton	3.5	1
London	3	1.2
Leeds	4.5	1.4
DOUBLE CONTACT		
Burnley	3	.3
Leeds	5.5	.6
Blackburn	5.5	.8
Sheffield	3.3	.8
Carlisle	4	1.1
Sheffield	3.3	1.2

Double contact, on the other hand, will produce a stable effluent, but one which is much inferior to that of a sprinkling filter or intermittent sand filter. On an average, primary contact will remove 50 per cent. of the dissolved impurity in the applied sewage, and secondary contact will remove about 50 per cent. of the organic matter in the effluent from the primary contact bed. Ordinarily, then, it can be assumed that from 70 to 80 per cent. of the

* Investigations on the Purification of Boston Sewage.

organic matter in the sewage applied to double contact beds will be disposed of. The effluents from contact beds are not always clear, although they may be. They are generally non-putrescible, but, bacteriologically, they are not good enough to discharge into streams which are used for public water supply, nor into tidal estuaries where there are oyster beds.

Effect of Temperature on Contact Beds—It is quite probable that contact beds can be successfully operated during cold weather in any locality where intermittent sand filters can be operated. The whole process occupies only a few hours in each bed, and there is not sufficient time for such a bulk of sewage to cool down perceptibly. However, the purification effected during the winter months will, no doubt, be considerably less than during a like period of warm weather, for the greater number of micro-organisms active in purifying sewage can thrive only when the temperature ranges between 50 and 100 degrees Fahrenheit, and, as in winter weather the temperature of the sewage will fall to from 35 to 40 degrees Fahrenheit, only a comparatively small number of bacteria will be active.

Double Contact Beds—Double contact beds are built with the secondary beds at a lower elevation than the primary beds, so that the effluent from the primary beds can discharge by gravity onto the secondary beds. Generally the application of sewage to contact beds is on the surface, although there is a tendency at the present time to fill the beds from below, and allow the liquid to rise in the filtering material, expelling the air from the voids. Until time and experience demonstrate the wisdom of that manner of filling contact beds, however, the better practice will be to apply the sewage or influent to the surface of the bed.

Automatic Apparatus for Contact Beds—Usually contact beds are operated by hand labor, as it is necessary to have an attendant at the plant who can operate the sluice gates. In some small plants, however, airlock apparatus is used to operate the gates, thus insuring a uniform period of fill, contact and draw.

SEWAGE IRRIGATION

Principles of Sewage Irrigation—In sewage irrigation, or sewage farming, as it is sometimes called, the crude sewage, after having been passed through a screen chamber and detritus tank to remove the coarser solids, is applied to the surface of ordinary agricultural land, to provide moisture for the growing vegetation, and at the same time to enrich the soil with the plant food which is carried both in suspension and solution by the sewage. When sewage irrigation is resorted to, purification of the sewage is usually secondary to irrigation, consequently larger areas of land are required than when purification of the sewage is the chief consideration. The large area of land required for sewage irrigation, however, insures a thorough purification of the applied sewage, providing the irrigation fields have been properly prepared for the purpose. The principles which underlie the practice of ordinary irrigation are what must be followed in sewage farming. There is no special treatment of the sewage required outside of straining. The method of preparing the soil, underdraining, flooding and cropping are the same, whether sewage or fresh water is applied. Raising of crops is the chief consideration in sewage irrigation, as it is in ordinary irrigation, and the fields, if prepared for that purpose, will completely purify the sewage which is applied, provided judgment is shown in the dosing of fields not to apply too much sewage at one time or at too frequent intervals. When the amount of sewage applied to a given area is not excessive, the organic solids are gradually dissolved with the formation of soluble products suitable for the food of higher plants, and the liquid seeps away to join the ground water, or is carried off in underdrains, as the case may be. If, however, the fields are overdosed, they become *sewage sick;* the surface clogs, pools are formed, putrefaction begins and a stench arises. Only a complete rest will then restore the fields to their normal condition.

Any soil that is suitable for agricultural purposes will be found suitable for sewage farming or irrigation.

SOILS

Composition of Soils—All soils suitable for farming are made up of varying proportions of sand, humus, silt and clay. No one of these ingredients alone makes a suitable soil, nor is it possible to find any one of these ingredients which is not mixed to a greater or less extent with the others. Loams, the most valuable of farming soils, are a mixture of the four ingredients. This mixture of different materials is beneficial not only in improving the texture of the soil, but also in providing suitable plant food, without which a soil would be barren. All of the soils from which crops are raised contain at least seven elements required for plant food; these are nitrogen, potassium, phosphorus, calcium, iron, magnesium and sulphur; without these no plants grow. Nitrogen, potassium, phosphorus and calcium are much needed by plants, and so the soil is liable to become exhausted of them if not fertilized. The iron, magnesium and sulphur are usually so abundant as to be practically inexhaustible.

Classification of Soils—The relative amounts of sand, clay and humus in a soil influence its texture, and serve as a broad classification for agricultural soils into sandy soils, clayey soils and humus soils. Besides these there are the loams, which are known as sandy loams and clayey loams.

Soils which contain 80 per cent. of sand and less than 10 per cent. of clay are known as sandy soils. They usually are leachy—especially if the sand grains are large—and are poor in plant food. Fine-grained sandy soils, as a rule, are better than coarse sandy soils.

Soils which contain from 60 to 70 per cent. of sand and 20 to 30 per cent. of humus are generally considered sandy loams, while soils which contain 70 to 80 per cent. of sand

and 10 to 20 per cent. of humus are considered light sandy loams. Usually, soils with an open, porous texture, like sand loams, are considered *light*, while soils which are of close texture, like clay, are said to be *heavy*.

Soils which contain 60 per cent. or more of clay or silt are commonly called clay soils. When the percentage of clay or silt reaches 80 per cent. or 90 per cent. the soil is worthless for farming. Clay loams are similar to clay soils, from which they differ chiefly in the less amount of clay they contain. A soil containing from 30 to 40 per cent. of clay is said to be a clay loam, while a soil which contains from 40 to 50 per cent. of clay is classed as a heavy clay loam. A clay loam usually contains from 25 to 35 per cent. of sand and a heavy clay loam contains from 10 to 25 per cent. of sand.

Loams, which are the most valuable farming soils, contain from 40 to 60 per cent. of sand and 15 to 25 per cent. of clay.

Voids in Soils—About 50 per cent. of the volume of ordinary soils is space, which is filled with air and water. Air is as necessary as water to agricultural soil. Nitrifying bacteria, which prepare organic matter in the humus, fertilizer or sewage for plant food, can perform their functions only in the presence of air or oxygen; therefore, a moist, well drained soil, of open texture, which permits a free circulation of ground air, is the best for bacterial activity, as well as for the growth of farm crops. The texture of soils varies much with the composition or the relative amounts of each material the soils contain. For instance, clay occupies about 65 per cent. of space, leaving only 35 per cent. of voids, while a sandy truck soil contains 37 per cent. of space, leaving 63 per cent. of voids. Soils of other textures vary all the way between these extremes.

The comparative fineness of different materials from which soil is composed can be seen in Table XI.

Soil Moisture—A good farm soil often holds more than one-half its weight of film water, after the free water has passed off. As a result of a force known as *surface tension*, each particle of soil holds a film of water over its entire surface, and no matter how well drained a soil may be, or how dry it might appear on the surface, this film is always to be found on the surface of the soil particles. Plants derive their nourishment from this film water, and not from the ground water or free water which sometimes fills the pores or voids of the soil. The moisture necessary to supply the surface films moves through the soil independent of the force of gravity, impelled by the force of capillary attraction or surface tension. Naturally, the precentage of moisture differs at different depths of the soil and at different points in the field. For instance, near the surface, where evaporation is rapid, capillarity cannot supply moisture fast enough to maintain an even distribution. Again, particles which are in contact with the fine hair-like roots of plants yield their film moisture readily to the plants. These particles, however, are in contact with other grains, which are not exposed to the capillary attraction of roots, and these latter grains yield part of their moisture to the drier particles, drawing in turn for a supply to replenish their films from the more moist particles with which they are in contact. In this manner soil moisture moves through the soil, up or down, crosswise or horizontally, independent of the laws of gravity and of the flow of ground water. Any moisture in excess of the film water is known as ground water. This excess water is not only of no value in soil, but is actually injurious, as it drowns vegetation.

To be valuable for raising crops, soil must be thoroughly underdrained, either naturally or artificially, to lower the ground water, so the soil and subsoil, as far as roots penetrate, will have only the film water to draw upon, and a plentiful supply of oxygen or air to prepare the plant food in the film water for consumption.

A coarse sand holds about 12 to 15 per cent. by weight of film water; a sandy loam from 20 to 30 per cent.; a clay loam from 30 to 40 per cent., and a heavy clay or a soil very rich in humus may hold 40 to 50 per cent. of film water. That is to say, a mellow loam, with a retentive subsoil, might hold from 5 to 6 inches of water in a foot of the top soil.

It will be noticed that the heavier a soil and the finer the particles the more moisture it will hold. That would naturally follow from the fact that if a 1-inch cube of stone be granulated, the grains will present more surface than the original cube, and the finer the stone is granulated the more surface it will present. While fine clayey soils hold more film moisture than do sandy soils, they cling to it more tenaciously and give up to the plants a much less percentage than do sandy soils. The liquid which enters the pores of a soil displaces the liquid or air which was previously present. The air is forced upward into the atmosphere and the water is forced downward to the underdrains or to the water-table. In order that sewage when applied to agricultural land will not pass directly through to the underdrain, the quantity must not be greater than can be taken up by the pores of the soil.

Owing to the greater capillary attraction of the small grains of clay over that of sand, the former will raise water from a greater depth than will sand. This strong capillarity tends to keep the soil moist and on this account the underdrains in artificial drainage should be nearer the surface when draining heavy lands than when draining light sandy soils.

Temperature of Soils—The temperature of soils has much to do with their value for cropping, and the temperature depends to a great extent upon their texture, composition, color and exposure. Sandy soils are warm because the large quartz grains hold heat well, and the coarser the sand the warmer it gets and the better it holds the heat. Clay, on the other hand, warms much faster than sand, because the particles lie closer together so that heat can pass more readily from one particle to another. For the same reason clayey soils lose more heat by radiation than do sandy soils, and, as they hold more water than sandy soils, they lose additional heat by reason of the greater amount of evaporation from the surface. So it is that fine-grained soils are colder than coarse-grained soils, although they absorb more heat; and sandy soils are considered warm, while clay soils are said to be cold.

Color of soils also has a thermal effect. Dark-colored soils absorb more of the heat rays of the sun than do light-colored soils of equal texture and composition; consequently, in northern latitudes, dark-colored soils are preferable to light-colored soils for irrigation or sewage farms. Fields on the southern slope of a hill, or on the level on the southern side of a range of hills, where they are protected from the wind and exposed to the sun, will be much warmer than fields with northern or western exposure.

Drainage of Soils—All soils used for raising crops should have good drainage, and unless the surface soil is underlaid by a porous subsoil of sufficient depth to ensure the removal of all water seeping through the upper layers, the soil should be underdrained with tile drains, if the fields are to be used for sewage irrigation. Underdrainage lowers the water-table, allows all surplus water to flow off so that film water, in the presence of air, will be available for the plants to feed upon. When the water-table is lowered, the roots of growing plants shoot downward, following the receding water level, and thus so much more available soil is added to the field. For instance, if the ground water in a field is within 14 inches of the surface, there are only 14 inches of soil for the roots to occupy and the available food and air are reduced almost to limits of barrenness. If, however, the field be underdrained so as to lower the water table to 4 feet below the surface, about three and one-half times the original growing soil will thus be made available. Underdrains not only carry off the surplus water from a field, but they also promote aëration of the soil.

Preparing Soils for Irrigation—Suitable soils of a light sandy texture, underlaid with porous strata of material to give good natural drainage, are not always available for sewage irrigation, and areas of land, such as are obtainable must be put in shape for the purpose. A sandy loam is the most desirable soil for irrigation. The grains being coarse, permit the ready passage of water through them after the capillary spaces have been filled, thus permitting the treatment of a large quantity of sewage. The soil is sufficiently retentive to store a supply of film water for plant food; capillary action is strong enough to raise water from the underground reservoir to the plant roots. Heat is given off very slowly, so the soils are warm; fields can be worked within a short time after a rainfall or after sewage has been applied; crops can be sowed at least two weeks earlier than in heavy soils, and the open texture of the soil permits of thorough aëration. There are two conditions, however, under which such soils are not suitable for irrigation; these are when the water-table is too high or when it is too low, and the remedy for either condition is underdrainage. When the water-table is too high, the excess of moisture reduces the temperature of the soil, excludes the air and dilutes the plant food, thereby retarding or stopping entirely the growth of the plants. Further, it submerges and renders inaccessible to the roots of plants great quantities of plant food stored in the subsoil, which can be reclaimed by lowering the water level. In arid climates when the subsoils contain considerable quantities of soluble alkali salts, such as sodium chlorid, sodium sulphate and sodium carbonate, and the water-table is very low, sometimes 40 to 60 feet below the surface, the salts become dissolved from the soils, and, in solution, are carried by capillary attraction toward the surface. If evaporation in such localities be rapid, the alkali contained in the water will be deposited in solid form near the surface, and the land will deteriorate for cropping.

Clayey soils are the exact antithesis of sandy soils, both in texture and in agricultural value. The very small spaces between the exceedingly fine grains of a clayey soil admit air and water very slowly, but hold the water tenaciously, so that when a clayey soil becomes thoroughly wet it is sticky. When the soil becomes dry it cracks, thus opening wide and numerous crevices, through which sewage can pass to the underdrains. If turned over by a plow while in a wet condition the soil bakes and becomes cloddy. When clayey soils are the only soils which are available for sewage irrigation, they should be treated to remedy the chief defect, which is heaviness. Underdrains will remove the surplus water, and promote aëration and warmth. Judicious cultivation will also do much to improve the texture of the soil.

The fine particles of clay can be separated from one another by mixing with humus, stable manure, green manure or sand, and plowing under deep. If sand or manure are not available, coal ashes mixed with the clay will give excellent results.

Systems of Underdrains—Underdrainage systems, for sewage irrigation fields, are similar in principles of construction to those for ordinary farm drainage. The object is to provide a system of underground channels large enough to carry off the maximum quantity of water discharged into them and to have the branch drains so spaced that the ground water between the drains will rise but slightly above the level of the tiles. The most economical system for thorough underdrainage is to have the branch drains parallel to one another, and discharge into a main intercepting line. On sloping ground, the branches may be laid up and down the slope, diagonally across the slope, or, where seepage of water moves laterally down the slope from above, an intercepting line may be extended along the upper edge of the slope to cut off the seepage.

The plan used in draining a tract of 480 acres of open black soil with clayey subsoil in Iroquois County, Ill., is shown in Fig. 52.*

This land was generally level but, before drainage, was dotted with ponds which contained water during six months of the year. The grades at which the drains were laid were in some cases 1 inch to 100 feet, and in others as high as 2 inches in 100 feet. Each line on the plan is designated by a name or number to distinguish it from the others, and its

* From Farmers' Bulletin, Drainage of Farm Lands, by C. G. Elliott.

length and size as well as its junction with other lines is indicated by the number of feet or the station number from

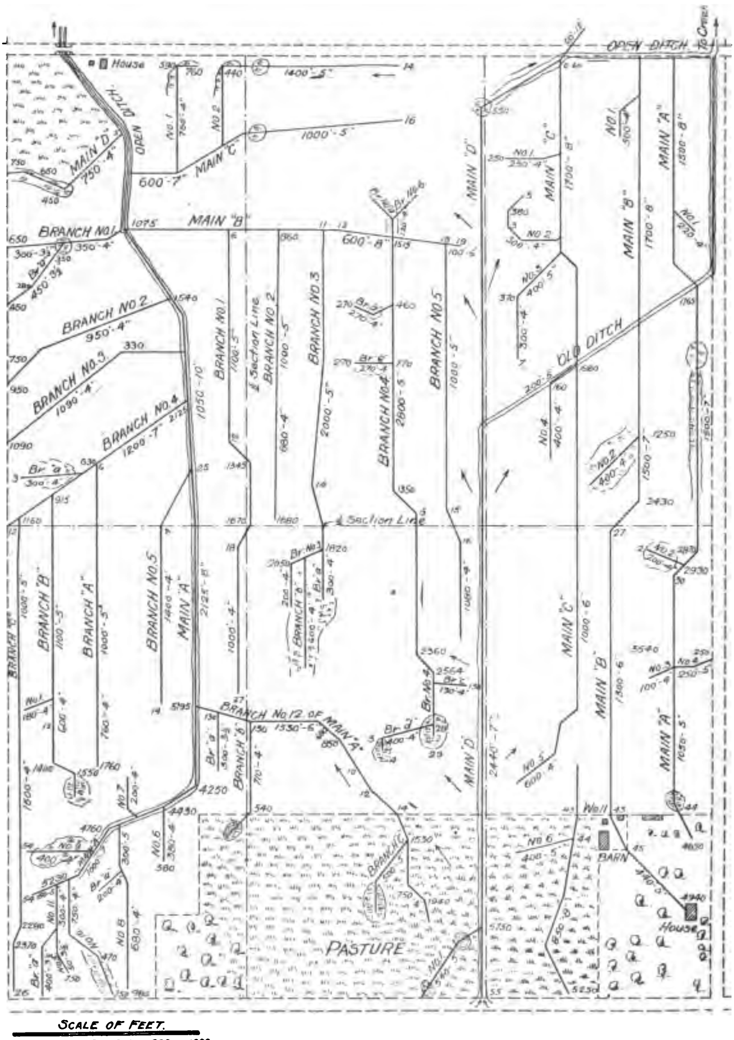


Fig. 52

the outlet point in each case. Such a map is valuable for the data it contains. When installing the pipes, everything is done according to system, previously studied out, while in case of a stoppage after the fields are put in service, the exact location of the drains can be determined without unnecessary digging. Another plan,* Fig. 53, shows the drainage of 128 acres of clay land in Jefferson County, Kentucky. By comparing this map with the one shown in Fig. 52, it will be noticed that there is a considerable difference in the systems, necessitated by the difference in the texture and composition of the soils. In the Kentucky plan, which was designed for

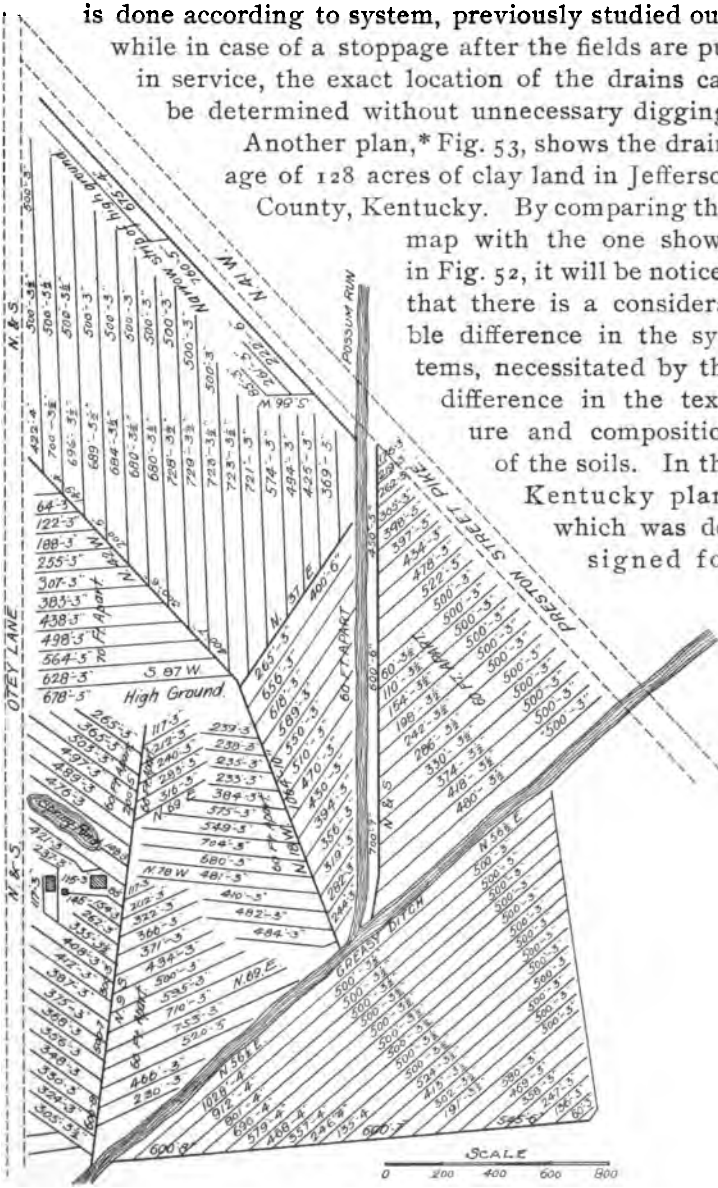


Fig. 53

* From Farmers' Bulletin, Drainage of Farm Lands, by C. G. Elliott.

clay soil, the drains are laid systematically 50, 60 and 70 feet apart, while in the Illinois plan, where the soil possessed better drainage properties, the system was designed with special reference to the natural drainage of the land and the particular requirements of unusually wet spots where pools of water stood for half the year.

For sewage irrigation work, when the soil is of approximately uniform texture, the plan of spacing the drains at uniform distances would probably be preferable.

Aëration of the soil is an important function of underdrains which is particularly important and of very great benefit to close soils. To promote aëration surface vents are sometimes connected to the drains to induce a more rapid circulation of air through the drains and soil.

Size of Underdrains for Irrigation Fields—The capacity of drains of different sizes and laid at various grades can be quite accurately calculated, but the quantity of water to be removed by a system of underdrains is a more difficult problem to solve. It may be stated, however, as a rule applicable to all underdrain systems for irrigation fields, that they should have sufficient capacity to carry off within twenty-four hours the excess water of the heaviest rains that fall. If they are made sufficiently large to care for the rainfall they will be large enough to carry off the small amount of effluent which seeps through to the underdrains. The soil is a great reservoir and will hold from 25 to 40 per cent. of its volume of water. In addition to this, evaporation* takes place rapidly from the surface of soil and leaves of vegetation, reaching even in moist climates the high volume of 70 per cent. of the rainfall. It would seem, therefore, that an underdrainage system, proportioned to care for the excess rain water, would be of sufficient size under any combination of conditions to care for sewage effluent. In practice, it may be assumed that one inch in depth of water must be removed in twenty-four hours from the entire irrigation field, and the underdrainage

* Each square foot of ordinary farm soil loses about 1.3 pounds of water daily by evaporation from the surface.

system should be proportioned to take care of that flow. If, however, the soil to be drained is a retentive one and in a locality not subject to heavy rains, allowance can be made to remove only ½ inch of rainfall. Three-inch tiles are the smallest that should be used. In Table XIII will be found the areas from which ¼ inch of water will be removed in twenty-four hours by tile drains of various diameters and different lengths when laid with different grades. Should the conditions be such as to require the removal of greater depths of water within twenty-four hours, a proportionate reduction should be made from the number of acres a pipe of certain size will drain. For instance, when laid at a grade of ½ inch in 100 feet, a 5-inch pipe 1,000 feet long will drain 17.3 acres of land of ¼ inch in depth of water, and $4\frac{3}{10}$ acres of land of 1 inch depth of water.

The size of underdrain pipe is increased as the system nears the outlet and receives the drainage of the larger area.

Depth of Irrigation Underdrains—The most advantageous depth for tile underdrains depends much on the characteristics of the soil and the distance apart of the drains. If shallow drains are laid wide apart in clayey soil, it is possible that midway between the drains the ground water will stand near the surface. In that case the obvious remedy would be to lay another drain midway between the lines already installed. The same result could be obtained by laying drains in clayey soils deeper, but 4 feet is the maximum depth that the drains should be laid in such soils, and 3 feet is perhaps a fair average. As a rule, drains should be placed as deep as they will readily receive water, no deeper. In sandy soils this depth ranges from 4 to 6 feet, with a fair average of 5 feet.

Distance Between Irrigation Underdrains—To secure efficient drainage, the branch lines of an underdrainage system should be placed sufficiently close together so there will be no appreciable rise in the level of the ground water between any two drains. The distance naturally depends on the composition of the soil, water traveling more freely

TABLE XIII*—SIZES AND CAPACITIES OF DRAIN PIPES

Diameter of tile in inches	Grade per 100 feet in decimals of a foot (with approximate equivalents in inches)											
	0.04 (½-in.)		0.05 (⅝-in.)		0.08 (1-in.)		0.10 (1 3/16-in.)		0.12 (1½-in.)		0.16 (2-in.)	
	Length of drain in feet.											
	1,000	2,000	1,000	2,000	1,000	2,000	1,000	2,000	1,000	2,000	1,000	2,000
	Acres of land drained.											
5	17.3	13.5	17.7	14.0	19.1	15.7	19.8	16.7	20.6	17.6	22.1	19.4
6	27.3	21.4	28.0	22.2	29.9	24.8	31.2	26.4	32.5	27.8	34.8	30.5
7	39.9	31.4	41.1	32.7	44.1	36.4	45.9	38.7	47.7	40.8	51.1	44.8
8	55.7	43.7	57.3	45.6	61.4	50.7	64.0	53.9	66.5	57.0	71.2	62.6
9	74.7	58.8	76.5	61.2	82.2	68.1	85.6	72.3	89.1	76.3	95.3	83.8
10	96.9	76.3	99.5	79.5	106.7	88.5	111.2	94.0	115.6	99.2	123.9	108.9
12	152.2	119.9	156.1	124.9	167.7	139.3	174.8	147.9	181.7	156.2	194.6	171.6
14	222.8	175.9	228.7	183.7	245.3	204.3	256.1	217.4	265.8	229.7	284.9	251.7
16	310.2	245.0	317.8	255.9	341.4	284.6	355.4	302.5	369.5	319.7	396.3	350.4
18	414.4	328.7	424.9	342.5	456.4	381.3	475.7	405.5	494.4	428.1	529.1	470.1
20	537.6	426.9	551.6	444.9	591.5	495.8	616.4	526.7	640.4	556.6	686.3	610.5

Diameter of tile in inches	Grade per 100 feet in decimals of a foot (with approximate equivalents in inches).											
	0.20 (2⅜-in.)		0.25 (3 in.)		0.30 (3⅝-in.)		0.40 (4¾-in.)		0.50 (6-in.)		0.75 (9-in.)	
	Length of drain in feet.											
	1,000	2,000	1,000	2,000	1,000	2,000	1,000	2,000	1,000	2,000	1,000	2,000
	Acres of land drained.											
5	23.5	20.9	25.1	22.7	26.7	24.5	29.5	27.5	32.0	30.3	37.7	36.3
6	37.0	33.0	39.6	35.9	42.0	38.6	46.4	43.5	50.5	47.8	59.4	57.3
7	54.3	48.5	58.0	52.8	61.6	56.7	68.2	63.8	74.0	70.1	87.1	84.1
8	75.6	67.7	80.9	73.6	85.8	79.0	95.0	89.1	103.3	98.0	121.4	117.3
9	101.4	90.7	108.4	98.6	114.9	106.0	127.0	119.4	138.1	131.3	162.6	157.1
10	131.6	117.9	140.6	128.1	149.3	137.6	165.2	155.3	179.2	170.5	211.1	204.4
12	206.8	185.6	221.1	201.8	234.5	216.9	259.2	244.1	281.8	268.6	331.8	321.7
14	302.5	272.2	323.5	296.1	343.5	318.1	379.7	358.2	412.9	393.9	485.8	472.1
16	420.6	379.1	449.9	412.2	477.4	442.9	527.8	498.4	573.7	548.8	675.2	657.3
18	562.2	508.1	601.8	552.5	638.1	593.7	705.2	668.0	767.4	735.1	902.3	880.5
20	729.2	660.3	780.0	718.2	826.9	771.1	914.7	867.8	994.5	954.6	1,170.1	1,144.1

* This table was computed by C. G. Elliott from the formulæ for determining the size for tile drains given in Elliott's Engineering for Land Drainage, which are:

$$(1)\quad v=48\sqrt{\frac{d(f+\frac{1}{2}k)}{l+54d}}$$

$$(2)\quad Q=av$$

$$(3)\quad A=\frac{Q}{.0105}$$

Where v=velocity of flow in feet per second.
a=sectional area of tile in square feet.
d=diameter of tile in feet.
f=total fall in length of drain.
l=depth of drain in feet at upper end.
k=total length of drain in feet.
Q=discharge of drain in cubic feet per second.
A=acres drained.

Constant 0.0105=quantity of water to be removed from 1 acre in 1 second of time.

Computations are made for two assumed lengths of drain—1,000 feet and 2,000 feet. $\frac{1}{2}k$ is 1.5 feet, that is one-half of depth of drain where the soil is open and saturated with water, under which conditions the drain will discharge its maximum quantity. Where the soil is close no additional head will be added by the free water of the soil, so that the factor $\frac{1}{2}k$ should be omitted in computations. Three feet of soil above the top of the drain has been assumed. It will readily be seen that the grade, length of drain and openness of soil are important factors in the capacity of a tile drain for discharging soil water.

through a light, sandy soil than through a close, tenacious clay. The common distances apart are 20 to 30 feet in very compact clay, 40 to 70 feet in average loams which have an open subsoil, from 100 to 200 feet apart in very open soils. A safe distance for drainage tiles in irrigation fields of average loam soils, is from 40 to 50 feet, if the depth is 3½ feet or over. Under the same conditions, but for heavy clay soils, a distance of 25 to 40 feet will be found safe.

Grades for Irrigation Underdrains—If underdrain tiles are laid at too low a grade there is danger of depositing velocities being attained which will silt up the pipe, possibly in course of time effecting a complete stoppage of the drain. If, on the other hand, drains are laid at too great a pitch, they will give equal trouble, but from a different cause. Twelve inches fall in 100 feet is considered about the limit of safety. If drains are laid at a steeper grade, the water is liable to attain such a velocity that it might loosen the tiles, particularly in a light soil. Three inches in 100 feet is about the right grade for underdrains. When necessary, however, they may safely be laid with as low grades as ½ inch in 100 feet, or as high as 8 inches to 100 feet.

The tiles should be laid with their joints as close together as possible, to prevent fine materials entering and obstructing the drains. There is no danger of making the joints too tight, for sufficient space will always remain to admit the ground water, no matter how close the tiles are laid. Most tiles are warped some in burning, and, when laying them on the bottom of ditches, the tiles should be turned and adjusted until they have not only tight joints but also a firm bedding from which they cannot be easily displaced.

METHODS OF IRRIGATING WITH SEWAGE

Ridge and Furrow Irrigation—One method of surface arrangement for irrigation by the ridge and furrow system is shown in Fig. 54. Ridges or beds of any convenient shape and size are laid out with furrows between. Sewage is discharged into these furrows, and, by force of capillary attraction, reaches all the roots within the beds or ridges. What moisture is not required by the roots at the time of applying, the sewage seeps into the subsoil, where it remains in storage ready to be drawn upon at any time. Main distributors *a*, which may be of pipe, puddled clay, wood, concrete or any convenient material, are used to distribute sewage to the various beds. At the various beds, sewage is allowed to flow through sluice gates *b* into a main furrow, from which the sewage is distributed to the various branch

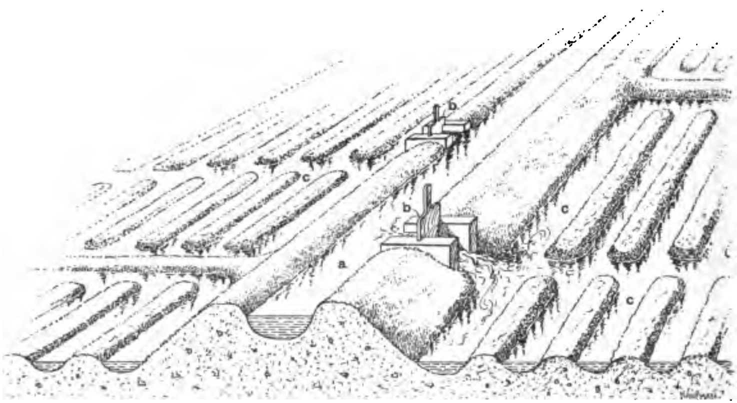


Fig. 54

furrows *c c*. Instead of having ridges and furrows, the surface of the soil may be made level, and the entire field flooded at frequent intervals. That would be the better method when the crops are of grass, nut trees or like vegetation.

Masonry abutments for the sluice gates are shown in this system of distribution. A slightly different method, in which wooden sluice boxes are used, is shown in Fig. 55. Wooden sluice gates or paddles are likewise used, as it is not necessary that they be absolutely tight. The main consideration in designing the distributing system for sewage irrigation is to secure durable, but inexpensive, flumes, sluices and gates.

Another system of ridge and furrow irrigation is shown in Fig. 56. This system is adapted to land having a slight slope which can be graded into a series of terraces. Sewage is applied to the soil in this system by flowing in a thin film over the surface from the distributing flumes or sluices *a a*, which occupy the crest of the ridges, to the collecting channels *b b* in the furrows, which in turn discharge into the main collecting channel *c*. This main collecting channel *c* answers in turn as a distributing main for the terrace next lower down, and so on, until the entire field is covered. In the illustration, the reference letters in the upper and

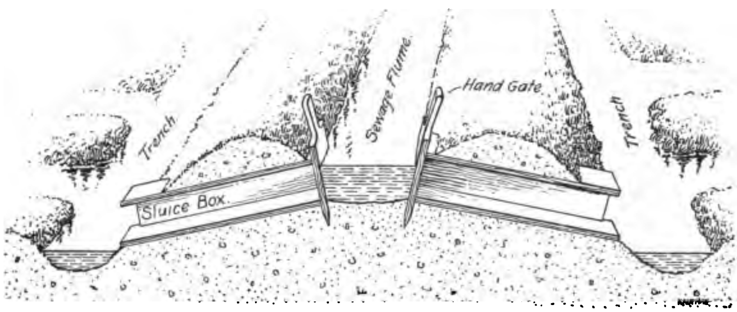


Fig. 55

lower terraces are reversed. The ridges in the upper terrace which are marked *a*, in the lower terrace are marked *b*, and the furrows of the upper terrace which are marked *b*, are marked *a* in the lower terrace. In irrigating by this method, the ridge channels are allowed to fill with sewage until they overflow their banks, and grooves are provided at certain intervals in the branch distributor, as shown at *d*, so sewage can be shut off from the ends of the ridges when sufficient sewage for irrigation has overflowed at these points, and the dammed-up sewage made to overflow other parts of the field.

In constructing irrigation fields according to this system, the beds are underdrained and are laid out in couples with slopes varying from 1 foot in 50 feet to 1 foot in 150 feet.

A clayey soil, being less porous than a sandy loam, would require a lower grade in order to absorb sufficient moisture while the film of water is passing over. Sandy soil, on the other hand, will wash easier, and the slope will have to be sufficiently low to prevent surface washing. Ordinarily the beds are laid out with a total width of 30 to 40 feet, half being on each side of the distributor, and a length of from 100 to 200 feet.

This method of laying out a field for irrigation is more expensive than the ridge and furrow method previously

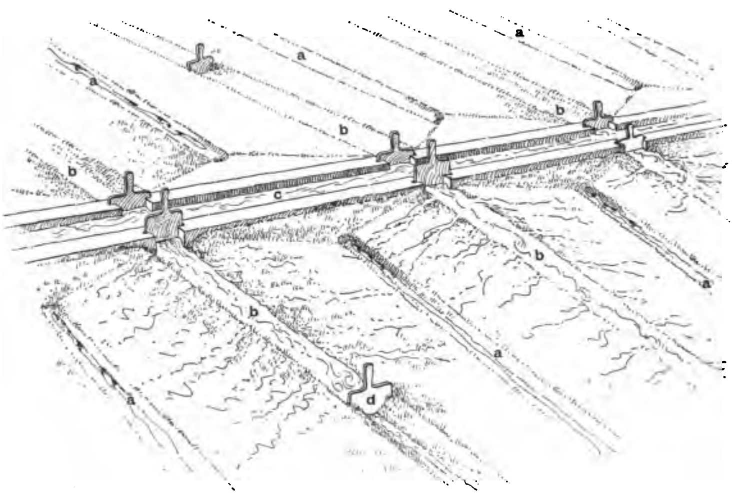


Fig. 56

shown, and when the simpler and less expensive method is practicable it is the better to use.

Catchwork System of Irrigation—A catchwork system of irrigation is shown in perspective in Fig. 57. In this system, which is specially adapted to steep hillsides, the sewage is delivered to the highest part of the area to be irrigated through a main carrier, which follows the contour of the land, and from which the liquid is caused to overflow the lower edge by damming the main distributor at various points. At various elevations lower down on the hillside are collectors or gutters following the contour. The sewage from the main distributor, overflowing from the lower side in a thin film, flows over the intervening space, and what liquid has not been absorbed by the soil is collected in the first gutter. When this collector is filled, the fluid overflows its lower edge as in the case of the main distributor and flows down to the next collector, and so on to the last gutter. The main carrier is given a fall of about 2 inches in 100 feet, so the farthest end of the field will be irrigated first; then by consecutive damming of the distributor, each part of the hillside will be successfully flooded. The main distributor is made from 8 to 10 inches deep and seldom over 24 inches in width. The gutters are made

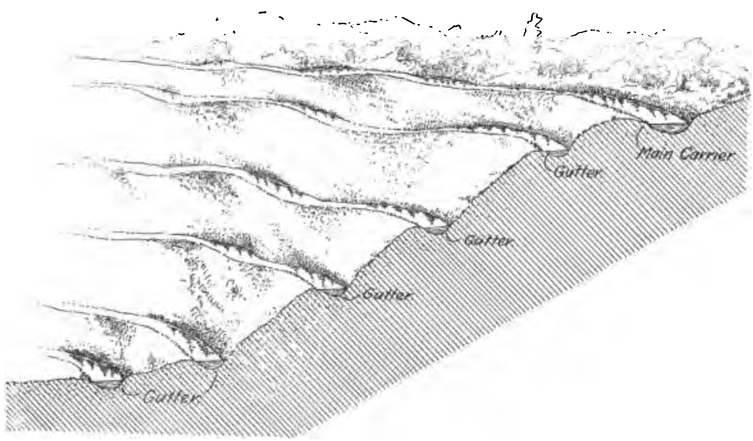


Fig. 57

level throughout their length and great care must be exercised in their construction so all parts of the area will receive their due proportion of water.

Quantity of Sewage Required for Irrigation—The quantity of sewage required for irrigation depends upon the kind of crops grown and the rate of evaporation from the surface of the soil. As the raising of crops is the chief consideration in irrigation, no more sewage should be applied than is actually required; if the limit of this requirement is exceeded the crops will be destroyed. In arid regions where evaporation is rapid, more sewage would be required than would be advisable for similar crops in a more favorable place. The application of sewage must be intermittent to allow the interstices of the soil to drain and aërate. An application on an average of from 5,000 to 15,000 gallons of sewage per acre per day is about all most crops will stand; allowing 100 gallons of water per capita as a daily average consumption, one acre of land at the foregoing rate would be required for from 50 to 100 persons. The general data concerning a number of British sewage farms are condensed into Table XIV. This table shows the high degree of purification of effluents which can be attained by sewage irrigation, the character of the sewage and rate of application for various soils that will give satisfactory results in practice.

Crops for Sewage Farms—It may safely be stated, as a rule, that any kind of vegetation indigenous to the locality can successfully be raised on sewage-irrigated farms. Among the crops successfully raised on sewage farms now in operation may be mentioned root plants, like carrots, parsnips, potatoes and turnips; legumes, like beans and peas; cereals, like oats, barley, wheat and corn; vegetables, like pumpkins and cabbage; soft-shell English walnuts, and forage, like alfalfa and Italian rye grass. The alfalfa, however, on account of its inclination to sod, is found difficult to cultivate. The raising of walnuts has given particularly good returns for the money invested.

SUBSURFACE IRRIGATION

Principles of Subsurface Irrigation—In the purification of sewage by subsurface irrigation, sewage is applied to the soil intermittently, by discharging the fluid in periodical doses into a specially constructed system of distributing tiles, buried in the earth, but laid close to the surface of the ground. The trenches in which the tiles are buried are filled with a coarse material like crushed stone, gravel or cinders, so that the sewage can easily escape to the

TABLE XIV—STATISTICS OF SEWAGE FARMS IN ENGLAND (R. S. C., 1904 A, B)

Farm	Date of construction	Average daily flow (million gallons)	Area of filter beds (acres)	Rate (million gallons per acre per day)	Character of soil and subsoil	Character of sewage	Method of treatment	Analyses (parts per million): Material	Total suspended solids	Nitrogen as—: Total organic	Free ammonia	Albuminoid ammonia	Nitrates and nitrites	Oxygen consumed in 4 hours at 80° F	Remarks
Aldershot camp	1864	1	120.5	0.008	Sand	Purely domestic.	Screening, settling tanks and land filtration.	Sewage	366	50.6	78.7	16.2	.....	207.9	Average of 3 sets of 24-hour samples, January, 1902.
								Effluent	.....	9.4	18.5	2.6	32.1	27.2	Average of 17 samples, 1900–1901.
Altrincham (Cheshire)	1870	.8	35	.023	Peaty soil lying upon sand and gravel.	Domestic.	Settling tank and land filtration.	Sewage	720	12.4	22.9	6.2	......	52.2	Average of 9 samples, 1900–1901.
								Effluent	.....	2.6	12.3	1.3	......	17.5	Average of 16 samples, 1900–1901.
Beddington (Croydon)	1861	4	420	.01	Gravelly loam overlying gravel and sand.	Almost purely domestic.	Screening, surface irrigation, and filtration.	Sewage	345	20.7	45	9.1	.....	124.8	Average of 7 sets of 24-hour samples, January, 1902.
								Effluent	.....	2.61	14.8	1.4	3.7	14.1	Average of 18 samples, 1900–1901.
Cambridge	1895	2.25	74	.3	Sandy loam overlying gravel and sand.	Mainly domestic; some brewery refuse	Screening, settling tank, and land filtration.	Sewage	265	18.8	38.2	9.1	......	10.8	Average of 3 sets of 24-hour samples, February, 1902.
								Effluent	.....	2.1	6.5	.9	12.6	8.1	Average of 18 samples, 1900–1901.
Leicester	1891	7.25	1,350	.005	Stiff clayey soil overlying dense clay.	Three-fourths domestic; one-fourth trade refuse.	Screening, settling tanks, surface irrigation and filtration combined.	Sewage	341	23.4	81.2	11.9	.....	223.5	Average of 3 sets of 24-hour samples, February, 1902.
								Effluent	.....	4.5	16.5	2	5.1	25	Average of 13 samples, 1900–1901.
Nottingham	1880	7	651	.011	Light sandy loam and gravel overlying gravel and sand.	Four-sevenths domestic; three-sevenths trade refuse.	Screening (partial) and land filtration.	Sewage	519	31.5	39.8	14.5	.....	232.1	Average of 3 sets of 24-hour samples, February, 1902.
								Effluent	.....	.8	1.3	.3	20.6	1.9	Average of 13 samples, 1900–1901.
Rugby	1867	.3	33	.009	Heavy loam overlying stiff clay.	Mainly domestic.	Screening, settling tanks, surface irrigation and filtration combined.	Sewage	473	32.9	61.1	17.3	.....	184.4	Average of 3 sets of 24-hour samples, February, 1902.
								Effluent	....	1.7	16.2	1.8	5.2	14.1	Average of 19 analyses, 1900–1901.
South Norwood	1864	.6	152	.004	Clay soil resting on London clay.	Purely domestic.	Screening, settling tanks, and surface irrigation (with a little filtration).	Sewage	219	14.9	35.4	6.7	.....	77.1	Average of 7 sets of 24-hour samples, January, 1902.
								Effluent	....	10.4	8.7	1	3.9	14.4	Average of 11 analyses, 1900–1901.

surrounding soil and there be attacked by the reducing micro-organisms in the presence of a plentiful supply of air. The tiles being located close to the surface and surrounded by porous materials, the aërobic bacteria in the upper strata of soil are the reducing agents, and, like in intermittent filtration, or surface irrigation, the process is more of an aërobic than anaërobic purification; consequently, the process is free from the putrefactive odors of anaërobic fermentation.

The surface of the ground over a subsurface disposal field may be covered with a top dressing of loam or garden soil and laid out as a truck garden, lawn, tennis court, flower garden, or may be put to any other ordinary use without interfering with the disposal process beneath.

Subsurface irrigation differs from surface irrigation simply in the method of applying sewage to the land. To successfully irrigate by the subsurface method, the land must be underdrained, unless there is a sufficiently low water-table; and the application of sewage must be intermittent, as constant application, even in a small stream, would saturate the soil without purifying the sewage. Sewage or effluent is discharged into the distributing system intermittently by means of an automatic siphon, located in a flush tank or dosing chamber, depending on whether fresh or septic liquid is to be applied to the soil.

The subsurface method of irrigation is the least satisfactory of any of the methods of irrigation where sewage is applied to land, but the method can be used without occupying space on the surface of the ground, for which reason it is preferred, in many cases, to better methods.

Sewage is applied to the soil in subsurface irrigation through drain tiles laid with open joints, at depths of from 12 to 18 inches, beneath the surface of the ground. The trenches in which the tiles are laid are filled in around the pipes with broken stones or gravel, to allow the liquid to flow freely from the drains and soak into a large area of ground. In a loose soil of open texture the liquid is quickly distributed by capillary attraction to all parts of

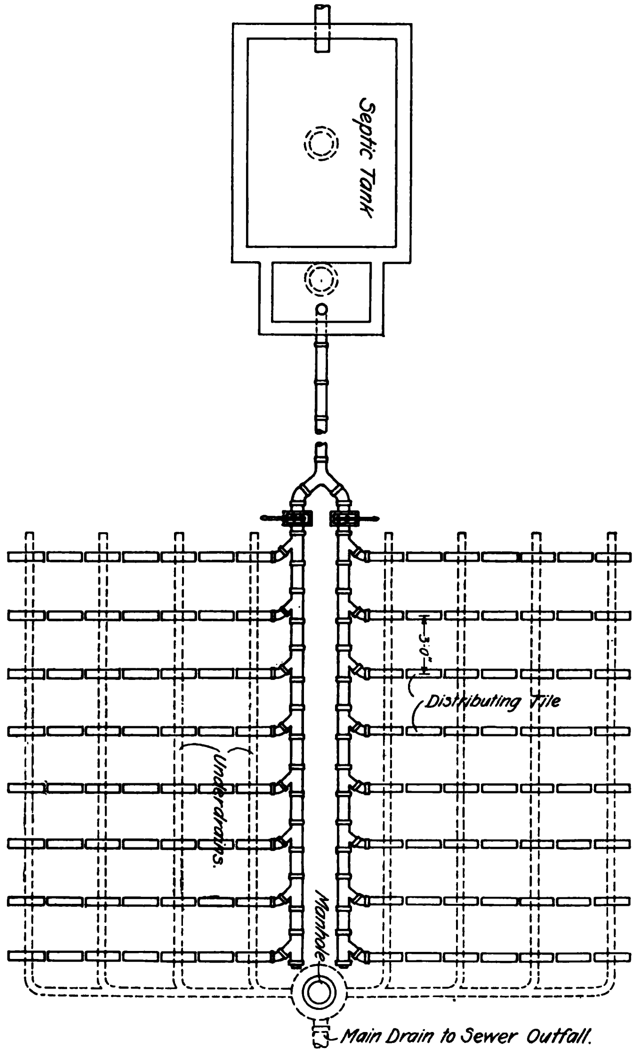


Fig. 58

the field between the lines of distributing tiles. It is a good practice when laying the distributing tiles to turn the end of each branch up and extend it to the surface of the ground to admit a plentiful supply of air to the tiles and voids of the soil.

Example of a Subsurface Disposal Plant—An example of a small subsurface disposal plant for level ground is shown in Fig. 58. This plant consists of two parts, the collecting tank and dosing chamber, and the disposal field. The disposal field is prepared for the purpose by suitably underdraining the soil, as shown by the dotted lines, and providing a system of subsurface distributors, through which to discharge the sewage into the soil. Any modification of this arrangement of pipes may be adopted. For instance, instead of having the distributing main at one side of the field as shown in the illustration, the main may be located in the center of the disposal field with branches taken off on both sides. In fact, that would be the better plan if the field were large. Again, instead of turning the branches at right angles to the main, as was done in this instance, they may be continued from the branches of the Y's at angles of 45 degrees.

Instead of running the underdrains at right angles to the distributing branches, they may be run parallel with them, in which case the underdrains should be spaced so as to come midway between two distributing branches instead of immediately beneath one. In case the underdrains are run as indicated on the drawings, the main branches should be extended to the sewer manhole with the cleanout plugs located within so the mains can be flushed, when necessary, direct into the sewers.

A septic tank is shown in this plant. A septic tank is not necessary, however, for subsurface sewage disposal. Equal, and in some cases better results can be obtained by using only a dosing chamber in which the crude sewage is collected until sufficient has accumulated to dose the field. An automatic siphon will then empty the contents of the tank into the distributing mains.

Whatever treatment the sewage undergoes before being discharged into the subsurface tiles, it should be aërated as fully as possible. Sewage, after standing for some time in a collecting or dosing chamber, also the effluents from septic tanks, are wholly devoid of air or oxygen, and as the succeeding operation is an aërobic one, the liquid should be discharged into the underground distributing tiles as fully charged with air as it is possible to have it under the circumstances.

In order that the disposal field itself and the interstices of the soil be freely supplied with air, the ends of the distributing tiles should be turned upward and extended to the surface of the ground. Indeed, a system of vents, so

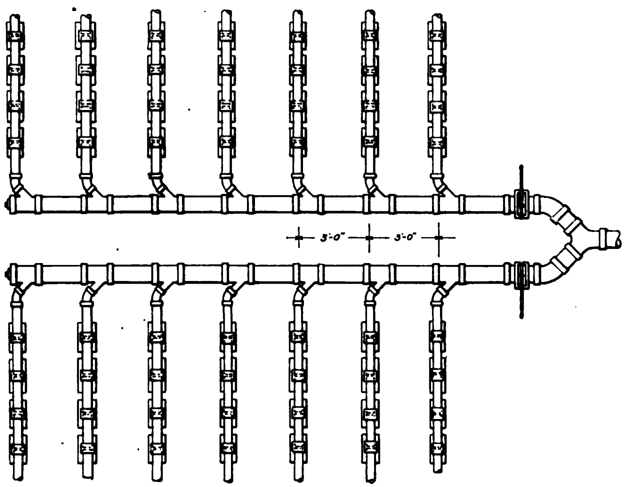


Fig. 59

arranged that air can circulate through the disposal tiles, will be found to increase greatly the effectiveness of the process and raise subsurface irrigation in effectiveness almost to the level of intermittent filtration.

Distributing System for Subsurface Irrigation—A section of the distributing system for the foregoing subsurface irrigation plant is shown in detail in Fig. 59. The main distributor is of salt-glazed tile, put together with water-tight cemented joints. Each distributing main is controlled by a quick-closing, lever-handle shear valve, and the end of the distributing main is plugged with a stopper which can be removed at any time to flush out the main.

The branch fittings for the distributing branches are made eccentric with branch outlets flush at the bottom, so that sewage can flow freely into all the branches, and the mains not stand part full of the liquid. The branch distributors are made of ordinary drain tiles 1 foot long and 3 inches in diameter. The tiles are laid with spaces of at least ¼ inch between the ends, which rest on earthenware gutters and are covered with earthenware caps to protect the joints.

Method of Laying Subsurface Irrigation Tiles—The method of laying subsurface tiles is shown in Fig. 60. A trench is dug for the line of pipe and the bottom of the trench is given a uniform grade of not more than 1 inch in 50 feet. On the bottom of this trench are laid the distributing tiles *a*, with open joints, which are protected from the entrance of dirt from the top by earthenware caps *b* and rest upon the gutters *c*. The caps and gutters are made with a larger radius than the drain tiles, so that sewage can escape freely from the joints into the trench, thence into the surrounding soil. In order that the sewage may move freely in a lateral direction, and thereby fill the entire trench to the normal level of the sewage, the trench is filled in around the pipes to within a few inches of the surface of the ground with crushed stone, broken bricks, gravel, cinders or any other substance which is suitable for sprinkling filters. The few inches of space above the coarse materials may be filled with a top dressing of any kind suitable to the purpose for which the surface will be used.

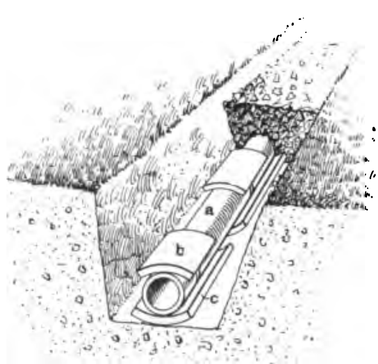


Fig. 60

Fittings for Subsurface Irrigation—Special fittings, which differ from ordinary sewer pipe fittings, may be had for subsurface irrigation. The most commonly used of subsurface irrigation fittings is the Y branch, shown in Fig. 61. It will be observed that the branch on this fitting is hubless and is taken off flush with the bottom of the main pipe, so that all sewage can drain from the main into the branch; further, the hub on the run of the fitting is on the opposite end from that of an ordinary Y fitting.

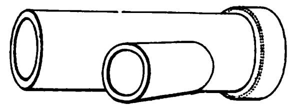


Fig. 61

Application of Subsurface Irrigation to Hillside—The principles of subsurface irrigation on a hillside are the same as on level ground, and in application the only difference lies in the manner of laying the tile. On a hillside, as on a level plain, the tiles must be laid almost level, and at the same time be within a certain distance of the surface. To accomplish this on hillsides, which have not perfectly plain surfaces, the trenches must be dug more or less crooked, following the contours, to keep them perfectly level and at the same time not buried too deep.

The application of subsurface irrigation to sloping ground with a plain surface is shown both in plan and in elevation in Fig. 62. It will be noticed that at every branch from the distributing main the line of pipe is stepped down a distance corresponding to the slope of the ground between branch tiles. In this case, the slope of the ground is but 1 inch in 1 foot, consequently at each fitting the main pipe is stepped down 3 inches, the branch distributors being spaced 3 feet apart. If, instead of stepping the main drain down, as shown in the illustration, it were sloped or given a grade similar to the surface of the land, the sewage would flow with considerable velocity to the lower end of the system and there break out through the trenches, thereby flooding the surface of the ground. By running the main horizontal, on the other hand, with a series of steps to keep it below the surface of the ground, the velocity of the liquid is retarded and the upper branches receive their shares of the sewage to be purified.

In installing a system similar to this on a hillside, the distributing tiles should be run near the top of the trenches, leaving plenty of space filled with crushed stone or other filtering material beneath but within 18 inches of the surface of the ground. If the distributing tiles were laid on the bottom of the trenches, there would be but little room for sewage, for as soon as the liquid rose above the level of the bottom of the branch fitting, there would be a head of water which would cause it to flow back into the main

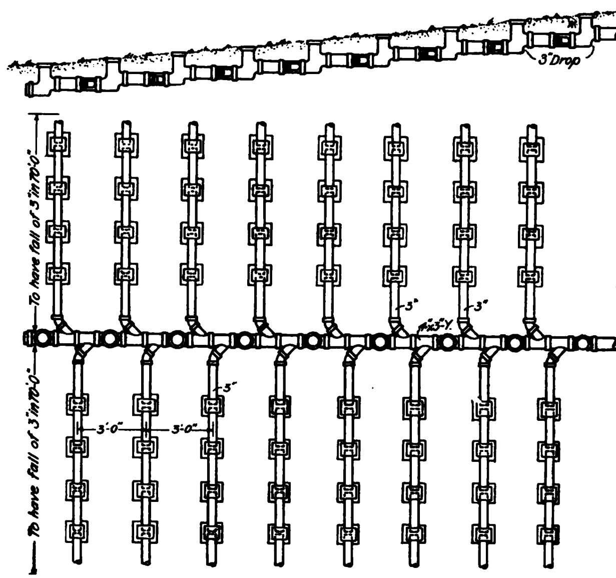


Fig. 62

drain, thence to the lower end of the field, which would become flooded. This system of subsurface irrigation may be put in without the branches of the step-down fitting extending to the surface of the ground as shown in the illustrations. Indeed, the branch which extends to the surface may be omitted altogether. However, where there is no objection to this method it will be found preferable, and, if the ends of the distributing branches are likewise turned up to the surface, there will be a circulation of air through the underground pipes and absorption ditches, which will make this process little less effective than surface application.

Size of Absorption Tiles—Absorption tiles for subsurface irrigation are usually 3 inches in diameter and 1 foot in length. Smaller pipes would be unsatisfactory on account of the greater liability of stoppage and because the air would not circulate so well through a system of small pipes. With pipes 3 inches in diameter, some air will be present even when the fields are being flooded.

Depth of Absorption Tiles—The nearer the surface of the ground that absorption tiles can be placed the better will be the aëration, consequently the better will be the purification of the sewage. In fields that are plowed, however, the tiles must be laid a sufficient depth below the surface so they will not be displaced by the plowshare. The greatest depth that absorption tiles are laid is 18 inches from the surface of the soil to the bottom of the drain tile. They cannot be laid in a trench much shallower than 8 inches without projecting above the surface, and, ordinarily, are laid at a depth of 12 inches from the surface of the soil to the bottom of the trench.

Grade of Absorption Tiles—Absorption tiles should be laid almost level, or at most at a grade of not more than 1 inch in 50 feet. If a greater grade is given the tiles, the sewage will be carried to the lower part of the system, where it might break through and flood that part of the field, while the upper portion remains comparatively dry. From a point about 25 feet away from the absorption tiles to the end of the drain the main drain pipe should be laid with but slight fall, so the sewage will not be carried by the momentum to the lower branches of the system to the exclusion of the upper branches. A fall of 1 inch in 30 feet will be found sufficient for the main drain.

Distance Apart of Absorption Tiles—Absorption tiles should be so spaced that there will be no appreciable fall in the water-line between any two branches and so the liquid seeping laterally will reach from branch to branch. In a heavy clay soil the lateral movement will be very slow, so that the tiles cannot be spaced far apart, while in an open sandy soil the downward trend of the sewage would prevent a strong lateral movement of the small quantity of sewage applied, so that in soil of open texture the distributing mains cannot be spaced far apart. In practice it is found that a distance of 3 feet will prove satisfactory for any soil suitable for subsurface irrigation Any greater distance apart would greatly reduce the size of dose that could be applied to a given area, while spacing the tiles closer together would greatly increase the cost of a plant without adding to its capacity or efficiency.

Area Required for Subsurface Irrigation—The area of land required for subsurface irrigation depends largely on the mechanical composition of the soil, its texture and whether or not the surface of the land will be used for any purpose. It is obvious that a coarse soil of open texture to which sewage can be applied twice or three times daily will require less area than will a fine soil of close texture to which sewage can be applied but three times a week. Further, if the surface of the land is used as a lawn or for any other purpose which would require that it be kept dry, less sewage could be applied per dose, consequently a larger area of land would be required than when surface wetting is not objectionable.

As a matter of fact, the area required, size of dose and character of soil are so intimately related that they must be considered together and in relation to one another. To obtain satisfactory results, every detail of a subsurface system should be worked out as carefully, or more so, than a surface irrigation plant, for in case of failure of subsurface irrigation there is danger of the many lateral branches becoming long, shallow cesspools. Because the results are hidden from view in this method of disposal, there seems to be more of an inclination to depend on empirical proportions than to work out the sizes and thus get accurate results.

Take, for instance, a retentive soil of close texture, such as No. 6 in Table IX, to which sewage can be applied but three times in seven days. If in such a soil the surface is laid out for a lawn, and surface wetting would be objectionable, no greater dose of sewage could be applied than would be contained by the absorption trenches without showing above the surface of the ground. An acre is a square of approximately 208 feet, and if distributing tiles were spaced 3 feet apart in an acre field there would be approximately 14,000 lineal feet of trenches and distributing branches. Allowing that part of the trenches which contains the gravel, stone or cinders to have a cross-section of 12 square inches, and the coarse material to occupy one-half the space, leaving 50 per cent. of voids, then one acre of such land could receive at one dose .5 x 14,000=7,000 cubic feet, or 52,500 United States gallons of sewage. But, from the mechanical composition of the soil, that is a greater quantity than could be cared for by intermittent filtration, which is limited to 34,000 United States gallons per dose, and, in subsurface irrigation, at least 10 per cent. less should be applied than to intermittent filter beds. If the soil were similar to that of No. 4 or 5 in Table IX, an acre would probably care for the full 52,500 gallons.

The foregoing is given merely to show the necessity for working out, from the character of the soil, the area of land required for subsurface irrigation.

In the absence of definite knowledge of the soil, empirical formula followed in practice when installing plants which have not been designed by an engineer is to allow, in porous soil, 1 foot of 3-inch tile pipe for each gallon of sewage, which is equal to 14,000 gallons of sewage per acre; and in clay soils 3 feet of 3-inch tile are allowed for each gallon of sewage, which is equal to 4,666 gallons of sewage per acre. According to the empirical formula, where water is used at the rate of 100 gallons per capita per day one acre of clayey land would be sufficient for not more than 50 people, while one acre of sandy soil would serve for 140 people.

Size and Frequency of Dose—The size and frequency of dose will depend upon the mechanical composition or texture of the soils, the same as for intermittent filtration or surface irrigation. Some soils will take a dose of 100,000 gallons or more per acre, applied three times in twenty-four hours, while other soils cannot be dosed with more than 34,000 gallons at a time, and those at intervals of not less than fifty-six hours. In order to properly design a subsurface system, a mechanical analysis should be made of the soils, if sand, or they should be classified as agricultural lands, if loams. Knowing the composition of the soil, the size of dose and frequency of application suitable for that soil can be found either in Table IX, which gives the quantity of sewage purified by sands of different mechanical compositions and the frequency with which sewage can be applied, or Table XIV, which shows the amount of sewage which can safely be applied to agricultural soils in sewage irrigation. Larger doses or more frequent applications than those stated in the tables should not be resorted to if satisfactory results are desired. Indeed, on account of the underground method of applying the sewage, better results will be obtained by allowing from 10 to 25 per cent. less for a dose.

The empirical method followed in practice, where no consideration is taken of the composition of the soil, is to allow for one flooding in twenty-four hours. When the size of dose used is small, this method will no doubt give fairly good results.

Underdrains for Subsurface Irrigation—Fields for subsurface irrigation should be underdrained the same as for intermittent filtration or sewage irrigation, unless there is a sufficiently low water-table to take care of the drainage naturally. For this purpose, the method of underdraining agricultural lands for sewage irrigation will be found satisfactory, and what has already been said on that subject will apply equally to the underdrainage of subsurface irrigation fields.

Operation of Subsurface Irrigation Plants—In operation, subsurface irrigation fields are dosed daily for one week. Sewage is then shut off from that field and it is allowed a period of rest of from one week to ten days, depending on the condition of the soil and the reserve irrigation fields. Usually two irrigation fields are provided, but where there will be a constant and uniform quantity of sewage during the entire year it will be found advisable to provide three fields. The surface of fields over absorption drains can be laid out as a lawn, flower garden or vegetable garden; no vegetation should be planted, however, the roots of which would be liable to reach down into the tiles and clog the pipes.

Subsurface irrigation is not seriously affected by frost, and can successfully be employed in regions where snow covers the ground during the winter months, provided the mean temperature of the air is not below 18 degrees Fahrenheit. This method of purifying sewage is not used extensively on a large scale for villages or cities, but finds its greatest application in the caring for domestic wastes from isolated houses where there are no sewer systems; hospitals, sanatoriums, prisons, asylums, country and seaside hotels, summer resorts and like institutions, located in country or suburban places. If properly designed and installed, subsurface irrigation will give very satisfactory results, but will be found more expensive to install than many other methods, and is liable to be the source of trouble and expense, owing to the clogging of the tiles by roots of plants, which enter the joints in search of water.

ANTISEPTIC TREATMENT OF SEWAGE

CHEMICAL PRECIPITATION OF SEWAGE

Principles of Chemical Precipitation—If sewage be stored in a tank for a certain period of time, or passed continuously through the tank, but at a low velocity, a certain proportion of the matter carried in suspension will be deposited on the bottom of the tank in the form of sludge, while particles of lighter specific gravity than water will float to the top. In that case the separation which takes place within the tank is due entirely to sedimentation, the particles having reached their various levels in accordance with the laws of gravity. With sewage, however, which contains substances differing widely in their specific gravities, the process of sedimentation is slow and inefficient, and can be increased, likewise the period of storage shortened, by adding to the sewage some substance which will produce a flocculent precipitant to hasten sedimentation. The precipitant in settling to the bottom of the tank gathers together and holds whatever suspended matter it encounters, thus effecting a partial clarification of the sewage. The precipitation of sludge by means of chemicals cannot be considered a purification process of the same rank as bacterial purification, for the entire organic content of the sewage still remains in the tank and must be removed or undergo the usual processes of fermentation and putrefaction before reaching a stable condition. However, a certain amount of clarification takes place, due to the mechanical separation of the suspended matter, and as the effluent contains less organic matter than the sewage, the process may be considered a purification. Usually the reagents used to precipitate sewage are of an antiseptic nature, and as the object of sewage purification is the ultimate destruction or resolution of the organic matter into other combinations, it is evident that an antiseptic process is the very reverse of the end to be attained.

Treatment of sewage by chemical precipitation usually consists of two stages; the precipitate must first be intimately mixed with the crude sewage, and the mixture must then be allowed a period of sedimentation in which to separate the sludge from the liquid. Chemical precipitation is not a suitable process for municipalities or country institutions only under exceptional conditions. Its chief claim to recognition lies in the value it might possess as a preliminary treatment for industrial wastes.

To be successfully treated by chemical precipitation, the sewage should be delivered at the precipitation works in a fresh condition. It is only the matter in suspension which is removed, and if the sewage has progressed so far that most of the organic matter is in solution, very little clarification can be effected. Liberal tankage should be provided for the sewage, and sludge should be removed from the tanks at frequent intervals and before putrefaction sets in.

Precipitants—Within certain limits, the results obtained by the use of a reagent depend upon the care and skill with which it is prepared, and the thoroughness and proportion of its admixture. There is a certain amount of any chemical used as a precipitant which gives the best results with the quality of sewage treated, and if less than that amount be used, or it be indifferently mixed, there will be less clarification, consequently less formation of sludge; on the other hand, if too great a quantity be used, and it be improperly mixed, some portion of the chemical will escape with the effluent. Many chemicals could be used as precipitants for sewage, but there are not many which fill the requirements of being chemically safe, non-injurious to fish, that will not greatly increase the amount of sludge, and that can be obtained in any locality at low cost. The selection of a reagent will depend greatly on the availability of the several suitable chemicals at the point of use. In Table XV will be found several chemicals and the results obtained by their use, singly and in combination, in experiments conducted by the Massachusetts

State Board of Health. The quantities of reagents used in the table are comparative, to show the clarification effected by using an amount of various chemicals, which at the time of the experiments would cost 30 cents per year for treating 100 gallons of sewage daily. As a matter of comparison, the results obtained by allowing the same sewage to settle for one hour, and the general results obtained by intermittent filtration are likewise included.

On account of the variation in the composition of different sewages, no one precipitant can be said to be better than another. Lime, which has been extensively used as a precipitant, is objectionable on account of the sludge it

TABLE XV—REAGENTS FOR PRECIPITATING SEWAGE

Chemical Used	Quantity of Sewage Treated Gallons	Period of Sedimentation Hours	Percentage of Albuminoid Ammonia Removed
1,800 pounds of lime . .	1,000,000	1	52
650 pounds of alum . .	1,000,000	1	51
1,000 pounds of copperas and 700 pounds of lime	1,000,000	1	57
270 pounds of ferric oxide in the form of ferric sulphate	1,000,000	1	59
Plain sedimentation . .	....	1	21
Filtered intermittently through 5 feet of sand .	....	..	98

forms. Alum sulphate and ferric sulphate give the best results under general conditions. The results with ferric sulphate are on the whole more satisfactory than those with alum sulphate. The effluents from sewage treated with iron salts, however, are slightly colored and under some conditions might be objectionable. It might be stated that, as a rule, ferric salts are preferable to ferrous salts by reason of their quicker action and more insoluble precipitate, and within certain limits the more of either ferric or ferrous sulphate used the better will be the result obtained. In adding chemicals to the crude sewage, a solution is preferable to solids. When the chemical is used in the form of a solution, care must be exercised to secure a thorough dissolution of all the substance in the mixing tanks, and to secure a uniform strength of solution. The variation of flow in sewage from hour to hour also calls for constant watchfulness on the part of attendants, unless some automatic device is used to proportion the amount of chemical to the constantly changing conditions. When the sewage flows into the settling tanks by gravity, the chemicals can be added to the sewage in a mixing channel before it reaches the tanks. The chemical may be discharged into the sewage through a number of perforations in a pipe extending across the channel, or from a notched trough from which the solution flows in a number of small streams. Usually a mechanical stirrer is provided to still further mix the chemical with the sewage. Sometimes a water-wheel, operated by the sewage, is used for this purpose, while in other cases baffle plates and deflecting boards are employed. In plants where the sewage must be pumped, the chemical can be introduced to the pump well, the action of the pump being depended on to thoroughly mix the chemical with the sewage.

EXAMPLE OF A CHEMICAL PRECIPITATION PLANT

Continuous-Flow Chemical Precipitation Tanks—In the continuous-flow process of chemical precipitation, crude sewage, to which a precipitant has been added, is allowed to flow slowly through a tank and discharge over a weir into the outlet chamber. A pair of continuous-flow precipitation tanks is shown in perspective in Fig. 63. In the sluice way to the tanks, baffle plates are placed so as to stir up and agitate as much as possible the chemically treated sewage. Each tank is provided with an inlet sluice gate, so sewage can be cut off from either tank. The inside of each tank is divided by means of walls or partitions into a series of channels through which the chemically treated sewage must slowly flow before it can reach the outlet weir. During this slow passage through the tank, the heavy particles of matter will be carried to the bottom, while those lighter than water will float on the top and be held back by the baffle wall built in front of the weir. The bottoms of the tanks are sloped toward one point, where are located

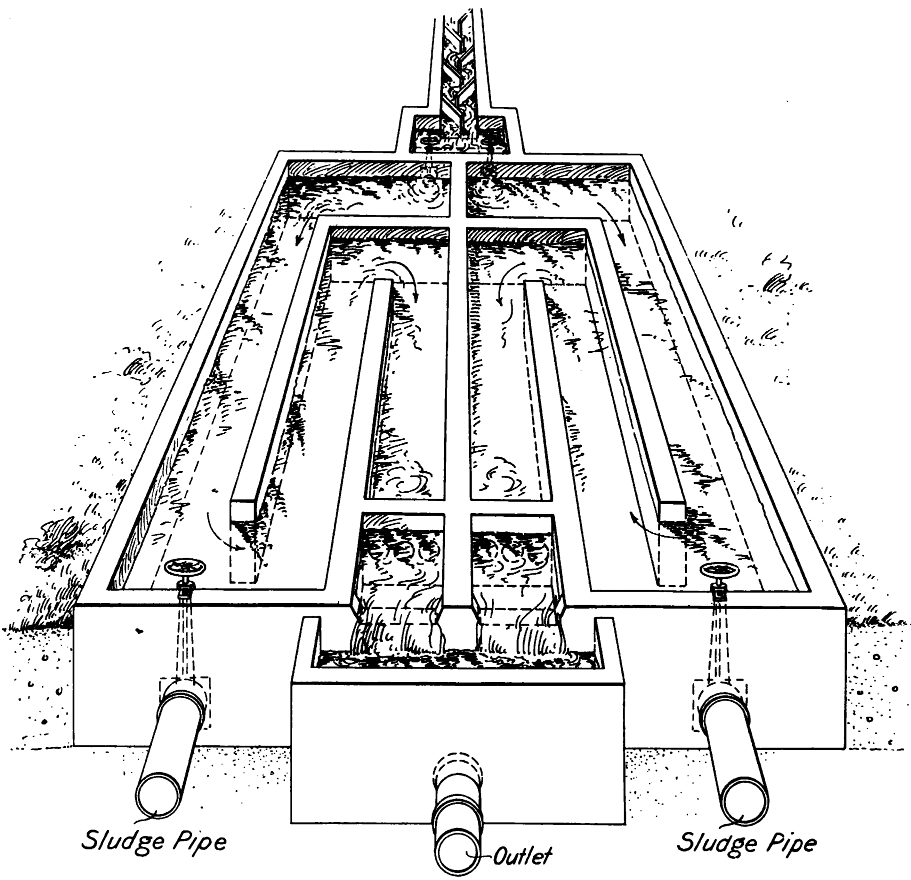


Fig. 63

valved sludge outlets, which may be connected to sludge wells, sludge beds or other place of sludge disposal. In case the sludge is to be handled by manual labor, and the tanks are at such an elevation that gravity discharge of sludge is impossible, the sludge pipes and sluice gates may be omitted.

In large chemical precipitation plants, manual labor is dispensed with as much as possible, and machinery is used to perform the heavy and laborious work. A building is necessary in which to house this machinery, and in the building provision must be made for the various apparatus. In the first place, tanks must be provided to mix the reagents in; sludge wells are required in which to drain the sludge; pumps are required for handling the sludge, and sludge presses or other devices for removing the water from the sludge so as to make it less difficult to handle. If sludge presses are used, air compressors and storage tanks will likewise be required for operating the presses, and instead of sludge pumps, compressed air sewage ejectors may be used for elevating the sludge from the sludge wells to the presses.

In short, there are many auxiliary parts that will be required for a fully equipped chemical precipitation plant. Just what those auxiliary parts will be is impossible to state, as they will depend in each case on the individual opinions of the designer. All that is necessary here is to point out the necessity for some kind of machinery for operating the plants and a building to house the machinery in. At some chemical precipitation plants, the tanks are housed in covered sheds, while in others they are exposed to the weather.

Fill-and-Draw Chemical Precipitation Tanks—Instead of the continuous-flow method, fill-and-draw precipitation is sometimes resorted to, although the continuous-flow method is generally considered the better of the two. In the fill-and-draw method, the sewage is treated with a chemical, stored in a tank for a certain period of time, seldom exceeding four hours, and is then withdrawn. If the continuous-flow tanks shown in Fig. 63 had their interior partitions removed, and draw-off sluice gates provided, they would answer very readily for fill-and-draw tanks.

Usually, chemical precipitation tanks are used in series.

That is, the sewage flows from the mixing channel into primary tanks known as "roughing tanks," where the heaviest particles, amounting perhaps to 75 per cent. of the sludge, settle, while the remaining 25 per cent. remains in suspension and is carried over the weir into the finishing tanks. In these finishing tanks the sludge which is carried in suspension from the roughing tanks settles, and the top water, which is comparatively clear, flows over the weirs into the effluent channels.

Floating Outlet for Decanting Effluents—Each chemical precipitation tank should be provided with a valved floating outlet, connected to the effluent pipe, so that the clarified liquid can be removed from the tank without disturbing the sludge. Then, when it is desired to empty a tank, the float effluent gate can be opened and the supernatant liquid drawn off through the floating outlet into the effluent channel. When the clarified water has been lowered to the surface of the sludge, the floating effluent gate can be closed, the sludge gate opened, and the sludge allowed to flow by gravity to the sludge well or sludge filters.

Capacity of Precipitation Tanks—Chemical precipitation is so little used in the United States that no definite proportions have been worked out for the sizes of tanks required. There should always be a sufficient number of tanks, however, so that one can be thrown out of service for cleaning and repairs without interfering with the operation of the rest of the plant. The size of tanks will depend somewhat upon the period allowed for sedimentation and whether rain water is discharged into the drainage system. In intermittent precipitation tanks, the period of sedimentation is approximately two hours, although four to six hours will effect greater clarification; and in chemical precipitation tanks where rain water is excluded, and one hour sedimentation provided for, a gross tank capacity of 40 per cent. of the daily flow will doubtless prove sufficient. The tanks should be deep enough so the precipitated matter will not be stirred up by surface currents and not so deep as to require a specially long time for the precipitated matter to reach the bottom. Tanks from 5 to 6 feet deep seem to fill both these requirements.

Sludge in Chemical Precipitation Tanks—The disposal of sludge is the most serious problem connected with chemical precipitation. Ordinarily, the wet sludge will amount to approximately 24 tons for every million gallons of sewage treated. The wet sludge contains approximately 5 to 10 per cent. of solids and 90 to 95 per cent. of water. After being air dried or passed through a filter press, the organic matter amounts to from 26 to 30 per cent.

At Worcester, the pressing and disposal of sludge costs over five dollars per million gallons of sewage treated, and to this amount must be added the cost of chemicals, labor and operation of the plant. Owing to the cost of chemicals the bother and cost of disposing of the sludge and the poor effluent which is liable subsequently to putrefy, this method is suitable only as a preliminary process to be followed by a biological treatment, and even then it is satisfactory only, as was previously stated, for treating industrial wastes.

SEWAGE PUMPING PLANTS

GENERAL CONSIDERATION

Many conditions arise in the sewering of cities or in the installation of purification plants, which necessitate the pumping of sewage. To do so, the sewage is collected in a receiving well or tank, from which it is pumped to the point of disposal. The wells are usually made in duplicate, and each well contains sufficient capacity for several hours' flow of sewage, while the pumps, unless they are operated by means of a directly connected motor, are cross-connected so that either or both can be operated by either or both of the prime movers.

Centrifugal pumps are found the most satisfactory for pumping sewage, and for this reason are generally used. Almost anything which will enter the inlet port of a centrifugal pump can be discharged through the force main, so that, as a rule, there is little trouble experienced with centrifugal pumps due to clogging. There is a notable exception to this statement, however. Cotton waste, when introduced to the drainage system, often finds its way to the centrifugal pump and interferes with its proper action. For this reason screens should be provided in sewage wells, to catch or hold back any large solid matter or fibrous material like cotton waste that might interfere with the operation of the pumps.

Centrifugal Pumps—Centrifugal pumps are best adapted for raising large quantities of liquid against low heads. For this reason they are found particularly suitable for pumping sewage. By the use of multiple stage centrifugal pumps, liquids can be raised to almost any reasonable height, but for ordinary work the multiple stage pump will not be required, the single stage proving entirely satisfactory. Single stage pumps, however, should not be used when the sewage must be raised a greater height than 150 feet. The open impeller pump is better suited to sewage pumping than is the closed impeller, notwithstanding the fact that closed impeller pumps possess about 4 per cent.

greater efficiency than equal sizes of open impeller pumps. Centrifugal pumps cannot be economically used when the quantity of sewage to be moved is less than 100 gallons per minute.

The United States Drainage Commission tests have shown that to obtain the best results the water entering centrifugal pumps should have a velocity of not over 8 feet per second and discharge velocities of about 12 feet per second. To produce those velocities in a well designed centrifugal pump, the impeller blades should have a circumferential velocity of approximately 50 feet per second. This will produce a whirl velocity through the discharge of the impellers of from 30 to 40 feet per second, which must be slowed down to 12 feet per second or less in the discharge pipe from the pump. To accommodate the various velocities in a centrifugal pump, the suction and discharge ports should be provided with taper connections, and no check valves not absolutely required should be used.

The submerged type of centrifugal pump is the kind best suited to pumping sewage. The pump is then located on the floor of the receiving well, and the shafting extends to the floor above where the driving mechanism is located. Pumps having discharge pipes of less than 2 inches diameter are not suitable for sewage work, and for municipal plants 2½ inches should be the minimum limit. It may be stated that the average commercial efficiency of the large size multiple stage turbine pump is about 65 per cent., and that of equal sizes of straight single impeller, volute, centrifugal pump, working against heads less than 100 feet, is 80 per cent. of the theoretical efficiency.

The efficiency of centrifugal pumps increases with the diameter of the suction and discharge pipes. For instance, the efficiency of a 2-inch pump is about 38 per cent.; of a 3-inch pump, 45 per cent.; of a 4-inch pump, 52 per cent.; of a 5-inch pump, 60 per cent.; of a 6-inch pump, 64 per cent.; of an 18-inch pump, 77 per cent., and of a 32-inch pump, 80 per cent. If, however, it is assumed that the size of pump used in average small installments will be somewhere between 3 inches and 8 inches in diameter, it can safely be assumed that the pump will not develop a greater efficiency than 55 per cent.

The capacity in gallons per minute of centrifugal pumps operated at different velocities may be seen in Table XVI. It should be remembered, however, that when designing a pumping plant the most economical velocity is approximately 12 feet per second, and the pump which will handle the required volume at this or a less velocity should be used. By so doing, not only will economy be obtained in the operation of the plant, but sufficient reserve pumping capacity will be available to handle any ordinary flow of sewage.

TABLE XVI—CAPACITIES OF CENTRIFUGAL PUMPS

Capacity per Minute Gallons	Size of Discharge Pipe Inches	Diameter of Pulley, Inches	Revolutions per Minute									
			6 Feet	8 Feet	10 Feet	12 Feet	16 Feet	20 Feet	25 Feet	30 Feet	35 Feet	40 Feet
200	1¾	6	425	590	680	725	825	900	975	1050	1120	1170
300	2	7	400	450	525	575	650	720	780	852	908	960
650	3	7	350	400	425	450	500	550	650	775	850	910
1250	4	10	275	300	350	400	450	500	600	675	800	890
2600	6	12	200	220	240	300	360	420	490	540	580	610
4750	8	15	185	200	225	250	310	360	390	425	450	475
7500	10	18	166	188	220	245	285	320	360	386	414	436

Methods of Driving Sewage Pumps—The method of driving sewage pumps must be determined in each case from the data at hand. If the pumping station be a small one, and worked intermittently, electricity might be advisable. If adjoining a power house where steam can be obtained at a reasonable price, steam engines might prove the most economical. When, however, the pumping plant is isolated from other buildings, as is usually the case at purification plants, it will be found necessary to have an engineer in charge, and under such conditions gasoline or kerosene engines will prove the most economical.

EXAMPLES OF PUMPING PLANTS

ELECTRIC PUMPING PLANT

The interior of a pumping station which is operated electrically is shown in Fig. 64. In this plant, three pumps are employed, each of which is located in a separate sewage well, and is direct-connected by vertical shafting to an

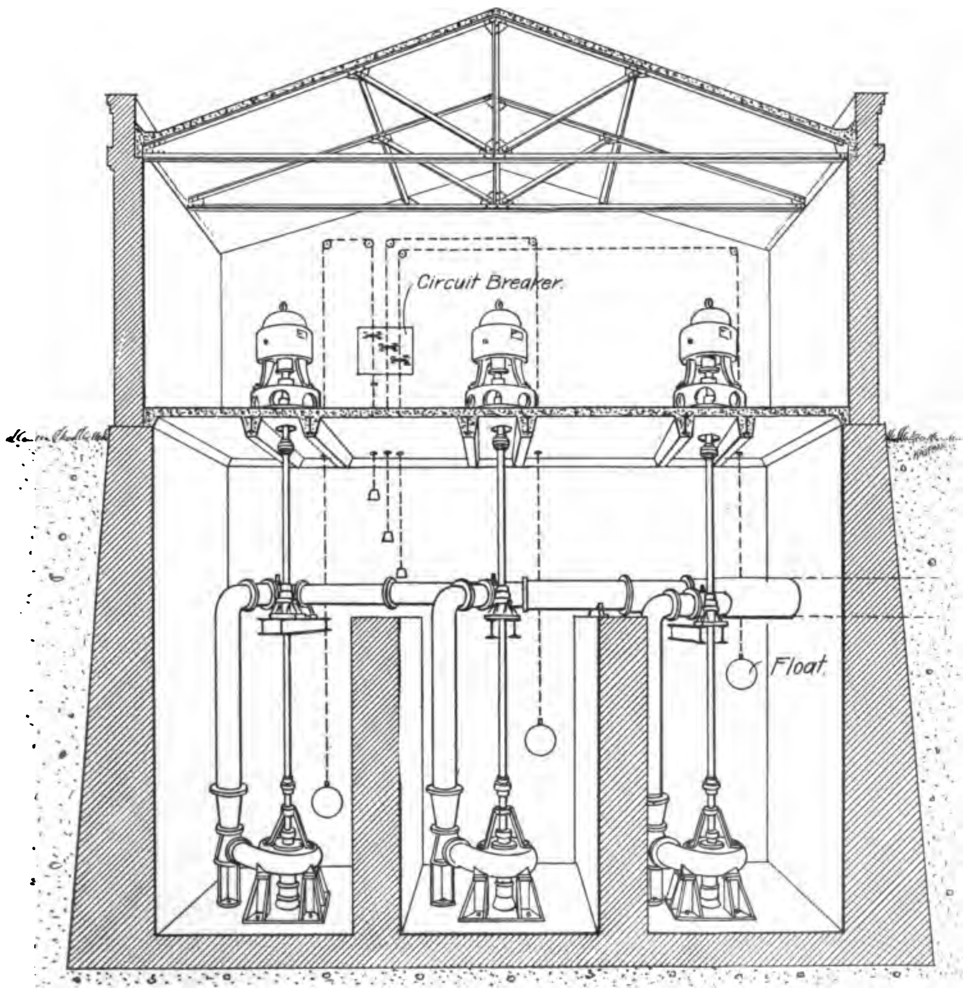


Fig. 64

electric motor in the room above. A slip coupling is placed in each shaft just below the floor and the weight of the motor and its shaft is carried by the motor bearing, while the weight of the pump shaft and the thrust of the impellers are supported by a thrust bearing, set on a pair of I-beams, which likewise serve as guides for the shaft. A slip coupling is found useful, as it permits the close adjustment of the impeller to the bottom of the pump casing, which is necessary to prevent the clogging of the pump by small rags of cotton waste winding around the shaft. Furthermore, it provides for the adjustment of the motor and thrust bearing, so that each will do its share of the work.

A thrust bearing is desirable in connection with vertical shaft direct-connected pumps, and one should be selected in which the oil is continuously and automatically circulated.

The pumps in such a plant can all be worked interchangeably or they may be adjusted to start when the sewage reaches different elevations in the several wells. For instance, the pump in the first well can be adjusted to start into operation when the sewage reaches an elevation of 5 feet; that in the second well when the liquid raised to 8 feet, and the last pump can be brought into service when the sewage reached the 10-foot mark. By this means, smaller units can be used, and ample reserve capacity will always be available to care for unusual conditions when there is an exceptional flow of sewage. Such a plan will be found very satisfactory for summer resorts or other places where the population varies greatly during the year, some months being several times as large as at other seasons. One of the pumps can then be used for the permanent population during the closed season and the other pumps brought into service as occasion requires.

The size, power and kind of motors required will have to be worked out independently in each case, and will depend greatly on the size of pumps and the heads against which they will have to discharge. It might be found advisable in some cases to provide a separate chamber for the float which operates the automatic switch to rise and fall in. When advisable, the chamber may be made of masonry, wood or iron, but must be open at the bottom to admit sewage. The inlets are not shown in the illustration, but the sewage should be passed through a screen of not larger than 1¼-inch mesh, for 6 to 8-inch pumps, before being discharged into the wells, and of not larger than 1-inch mesh for smaller pumps.

Gasoline Pumping Station

Where a large quantity of sewage must be handled daily, the constant supervision of an engineer will be necessary to look after the machinery, so that the automatic apparatus can be dispensed with. Under such conditions, gasoline or kerosene engines will be found the least expensive to operate, and, everything considered, will probably be the most satisfactory forms of prime movers.

The interior of a gasoline or kerosene engine pumping plant is shown in perspective in Fig. 65. In this plant there are two separate wells formed by a partition built in the center of the large circular basin. In each well is located a submerged type of centrifugal pump, with a vertical shaft terminating in a bevel gear above the floor. The two pumps are wholly independent of each other, so that one cannot be used to pump sewage from the other well. Further, each pump is valved so that when not in use, or when disconnected, the discharge pipe can be put out of service by closing the valve. By this means, either pump can be operated separately, or both pumps can be operated at the same time.

The inlets to the two wells are valved, so that sewage can be cut off from either; and before reaching the sluice gates, the sewage must pass through a screen, to remove all substances which would clog or otherwise interfere with the operation of the pumps.

There are two engines used for operating the pumps. Nominally, one engine is intended to operate one particular pump, but the engines are so connected with shafting that

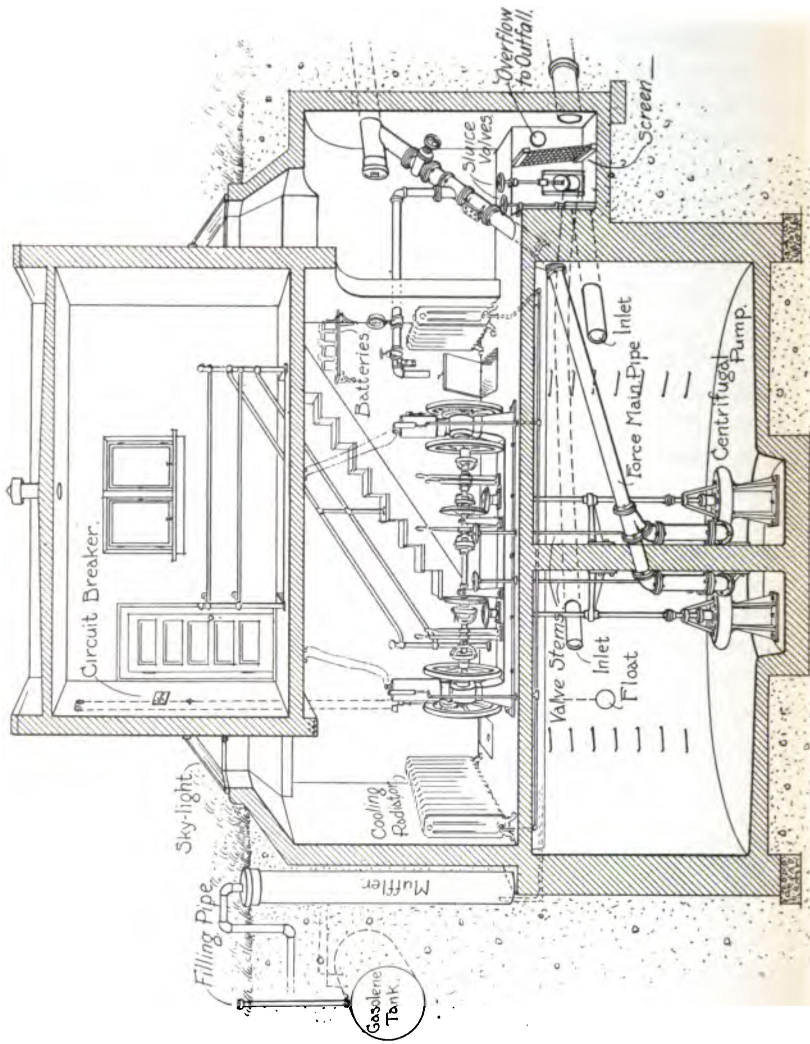


Fig. 65

either engine can operate either or both pumps, or the two engines can work in unison, giving a common impulse to the horizontal driving shaft.

Wherever internal combustion engines are used, provision must be made to keep them cool. This is usually accomplished by circulating water through a water-jacket casing around the cylinders, and passing the heated water through a radiator to cool. Such a method of cooling necessitates a supply of cold water and means for circulating it through the cooling system. Batteries are required for the ignition system, and a good muffler, while not indispensable, is at all events advisable, as it deadens the noise of the explosions in the cylinders. The fuel tank may be situated at any convenient point about the premises, and need not be at a higher elevation than the engines, as the gasoline is drawn into the cylinders by suction. If buried in the ground, the tank should first be covered with a coating of some good preservative, such as asphaltum, to protect the metal from rusting.

The foregoing illustrations are offered only as suggestions, to show the application of electricity and internal combustion engines to sewage pumping. Any other type of machinery may be used for the same purpose, the chief considerations being efficiency and economy.

DISPOSAL OF SEWAGE

DISPOSAL OF CRUDE SEWAGE BY DILUTION

Disposal into Tide-water—Cities situated on the shore of an ocean often solve the problem of sewage disposal by discharging the crude sewage direct into tide-water, depending on the ebb tide to carry all solid matter out to sea, while the vast dilution of the large body of water sufficiently cares for the matter in solution. This practice has not been objectionable in the past, when the volume of sewage was small, or when a large quantity of sewage was discharged into tide water in small quantities from numerous outfalls, advantageously situated along the shore line. As communities grow, however, the problem increases, and it is doubtful if at the present time it is either advisable or permissible for large centers of population to so dispose of their sewage. This method, which, under favorable conditions, may be suitable for a city of 10,000 inhabitants, might be hopelessly inadequate for a community of half a million people. Further, if the outfall is situated in a protected spot, or the city is on the shore of a protected bay, continued discharges of sewage into such a harbor are liable to cause a nuisance. In the vicinity of beaches used for bathing, discharging the sewage into tide-water would be not only a disagreeable nuisance, but might prove a source of disease, while in the vicinity of oyster or clam beds such a practice would be attended with grave danger.

In the past, when no adequate method of purifying sewage was known, the practice of disposal into tide-water was permissible, and in a manner solved the problem of sewage disposal. When a large number of people are grouped together, an unusual or abnormal condition is created; vast quantities of organic matter in the form of foodstuffs are shipped to the community, and some method of disposing of the refuse from such foodstuffs must be devised. Discharge into tide-water seemed a happy solution of the problem in former times, but the large cities on the seacoast are now turning their attention to purifying the sewage before discharging it into the water; and what is found desirable for large cities will apply likewise, although in a less degree, to small cities situated on the seashore. A less degree of purification is required for sewage which is discharged into the ocean than would be required for an effluent which is to be discharged into a stream or small fresh water lake, unless the sewage is discharged into the ocean near shell fish beds, in which case an extra degree of purification might be desirable. Ordinarily, however, if the process of purification has reduced the easily putrescible bodies, the more stable or slowly decomposing matter may be discharged into tide-water.

Disposal into Fresh Water Lakes—The practice of disposing of crude sewage into fresh water lakes is more objectionable than disposing of it into tide-water. The water from lakes is used for water supplies, and the entire water supply for a community might become infected from sewage and cause an epidemic of disease in the town supplied with the water, unless it is filtered before being delivered to the consumers. When sewage is discharged into fresh water, the outfall should be at least one-half mile from the shore, well away from a waterworks intake, and the sewage should be discharged from a number of outlets along the outfall sewer, in order to dilute the sewage to a high degree in the shortest possible time.

Disposal into Streams—It is commonly supposed that flowing streams into which sewage has been discharged have the power of purifying themselves, and that a few miles below the point of contamination the water will be as pure as before receiving the sewage. This power of water to cleanse itself, which is known as the self-purification of streams, is more limited than is generally supposed, and depends for its operation on the presence of a suitable supply of oxygen and sufficient aërobic bacteria. So long as the dissolved oxygen in a stream is greater than 25 per cent. of saturation, the nitrifying bacteria can perform their life's work, but should the proportion of organic matter in a stream be increased so that the percentage of saturation falls much below 25, the conditions are changed, and instead of aërobic decomposition, anaërobic putrefaction takes place, foul smelling gases are produced and the stream becomes an open sewer; even when the percentage of saturation falls below 50, at times, the water is liable to become offensive. Evidently, then, there is a limit to the quantity of organic matter which can be purified in a running stream. This limit has been variously estimated in the case of sewage by different authorities, who allow a certain number of volumes of water to each volume of sewage. These estimates, however, do not take account of the difference in the strength of different sewages nor the condition of freshness or staleness when discharged into the water, so that at best they are merely approximations.

The proportion of sewage which, according to various authorities, can be discharged into streams with safety may be found in Table XVII.

TABLE XVII—SAFE DILUTION OF SEWAGE IN STREAMS

Authority	Nuisance Probable	Nuisance Improbable
Pettenkofer	. . . .	1 of sewage to 15 of water
Stearnn	. . . .	1 of sewage to 40 of water
Herring	1 of sewage to 16 of water	1 of sewage to 45 of water
Goodnough	1 of sewage to 23 of water	1 of sewage to 36 of water

Dilution of sewage is not of itself a purification, but merely an aid to purification. By separating widely the particles of organic matter in the sewage, dilution enables the micro-organisms in the water as well as the larger microscopic forms of animal organisms, the crustacea, rotifers and protozoa, to carry on their work of reduction in the presence of an abundant supply of oxygen. It is due to these microscopic forms of life that the self-purification of streams is effected. Self-purification of streams is not by any means a rapid process, and the distance below the point of pollution where the water regains its original purity is a matter of doubt. The River Pollution Commission of Great Britain (1874) concluded that sewage mixed with twenty times its volume of water would be only two-thirds purified in flowing 168 miles at a rate of 1 mile an hour.

The water of streams with a sluggish flow will be purified in a much shorter distance than will streams with a high velocity. This is due to the fact that sedimentation is greater when the flow of a stream is at a depositing velocity than when the current is swift enough to carry suspended matter along. The additional time in which the reducing organisms have to work is an additional factor in promoting clarification. Sedimentation, however, is not a purification, and although the lower reaches of a river below the point of sedimentation are purified, the deposits of sediment foul the river bottom, kill or drive away fish, and in course of time create a nuisance in the stream. Such is the condition of the Passaic River in New Jersey, which at present is a large open sewer, so foul that in its water fish cannot live.

DISPOSAL OF SLUDGE

Composition of Sludge—Sewage sludge is the solid residue which settles to the bottom of septic and sedimentation tanks or other receptacles used in sewage purification. According to the experiments of the Massachusetts Board of Health, sewage carries in suspension one part of solid organic matter in every thousand parts of water, and this solid part of organic matter unites with nine hundred and ninety-nine times its bulk to form sludge.

The composition of septic tank sludge differs from chemical precipitation sludge both in quantity and in chemical composition, and the composition of chemical precipitation sludge, also septic tank sludge, depends on the composition of the sewage, the amount of storm water which enters the sewers and how carefully the sewage is screened before entering the tanks. Ordinarily, the sludge in a septic tank consists of about 90 per cent. water and 10 per cent. solids, a large percentage of the solids being inorganic matter.

Sludge is the most objectionable and at the same time the most difficult part of sewage to dispose of without creating a nuisance and the method of disposal will depend to a great extent on the location of the plant and amount of sludge to dispose of.

Disposal of Sludge in Deep Water—When purification works are situated close to the seashore, the sludge from tanks can be disposed of by running it onto dumping scows which are towed out to deep water where the sludge may be dumped. Whether it would be good practice to dump sludge in the Great Lakes is doubtful, but if the practice should be resorted to, the scows should be unloaded well away from the intake to a water supply system, and a sufficient distance from shore so as not to foul the beaches.

Burning of Sludge—Sludge filters of coarse material may be prepared, and the surface of the bed covered with hay, straw or other combustible material, after which the sludge may be pumped or allowed to flow by gravity onto the sludge beds. After the water has seeped away and the remaining moisture evaporated, the sludge and straw or hay can be forked out, placed in a heap and burned. If there is no sludge bed at the plant the sludge can be mixed with combustibles, such as peat, tanbark or sawdust and disposed of by burning. Sludge may also be burned in a garbage destructor, as is done in several plants where garbage destructors and purification plants are combined.

Sludge Used as Fertilizer—When sludge is to be used as a fertilizer it may be run or pumped onto agricultural fields and plowed under. It may be mixed with earth, loam, vegetable mold, leaves, grass, stable manure, ashes, sawdust or any other suitable material, and piled in compost heaps for future use; or the sludge may be deposited in large open basins, surrounded by an embankment and left for the moisture to evaporate, after which it can be carted away to use for fertilizer or to fill low lands. When sludge is used for filling low lands, each application of sludge to the soil should be covered with a layer of earth, ashes or like material. Ordinary sludge from settling tanks cannot be spread upon land without creating offensive odors, while sludge from septic tanks can be disposed of in that manner without being offensive. The sludge from the settling basins receiving the effluent of sprinkling filters is much less offensive than ordinary sludge and can be applied to land with practically no nuisance.

Sludge may also be mixed with lime, then compressed into cakes in filter presses, and in this form, which makes it easy to handle, conveyed to accessible places for fertilizer or for filling material. The value of pressed sludge, however, seldom equals the cost of the lime and pressing. While the sludge possesses no greater fertilizing value than an equal weight of barnyard manure, on the other hand it is open to the objection that it does not give good results as a fertilizer when applied continuously year after year to the same piece of land.

Some sewage sludges are so rich in fats that it is practicable to recover the grease or use it as fuel. The separation of grease from sludge does not deprive it of its manurial value, as the nitrogen is not extracted.

Disposal of Sludge on Land—There are two methods of disposing of sludge on land. In the first method, the sludge is pumped or otherwise discharged on to land properly embanked to a depth of 8 or 10 inches, and the water allowed to disappear by percolation and evaporation, leaving the dry residue, which may be covered with a thin layer of earth.

In the second method, the ground is dug in a series of long parallel trenches, 3 feet wide by 18 inches deep, with a space of 3 feet between trenches, and the excavated material is piled up on the spaces between. A large main trench is dug at right angles to the sludge disposal trenches and the liquid sludge is pumped into this main trench, from where it runs to the branches.

The supernatant water and thin sludge are first pumped into the trenches, and allowed to fill the lower ends, the thicker sludge being pumped in after the greater part of the water has become absorbed by the bottom and sides of the excavations. Small earthen ramparts are left in the trenches at regular intervals of about 50 feet, to intercept the solids and allow the water to run off in front.

In about three days after the trenches are filled, the sludge is sufficiently set so a light covering of screened soil about 1 inch thick may be spread on top. After a further interval of ten days, the sludge is generally sufficiently consolidated so that the trenches may be filled with the earth, which was originally excavated.

In the course of a few weeks after the filling of the sludge trenches it is generally practicable to excavate intermediate trenches from the solid ground which was originally left between the lines of the first trenches.

Once an entire field has been covered with sludge in this manner, it is advisable, under the most favorable climatic conditions, to allow the field to rest for at least two years before re-sludging. This will necessitate at least three sludge fields, to be used in rotation, in a three year cycle. A soil of open porous texture with low water-table is the best suited for sludge fields.

Pressing Sludge— Sludge cannot be easily pressed without the addition of lime or some other substance to give it body and at the same time act as a binder for the particles of sludge. On account of its comparative low cost, value as a fertilizer, and the ease with which it can be procured, lime is generally used for this purpose.

The caking qualities of sludges vary considerably, in fact more than would be expected. For instance, septic sludge is more difficult to press than either chemical precipitation sludge or sludge from plain settling tanks, and at the same time requires more lime to form a cake, besides necessitating more care in operating the presses.

Special presses, one of which, the Shriver, is shown in Fig. 66, are required for the pressing of sewage sludge. In operation, after the sludge has been mixed with a certain proportion of lime, it is forced into the filter presses by means of pumps or compressed air, under a pressure of 60 pounds per square inch. This pressure is sufficient to force the water through the filter cloths, while the solid matter is arrested and retained in the chambers of the filter press, forming cakes anywhere from 1 inch to 3 inches thick, according to the method of treatment. When the presses are full, that is, when hard cakes are formed, the filtrate

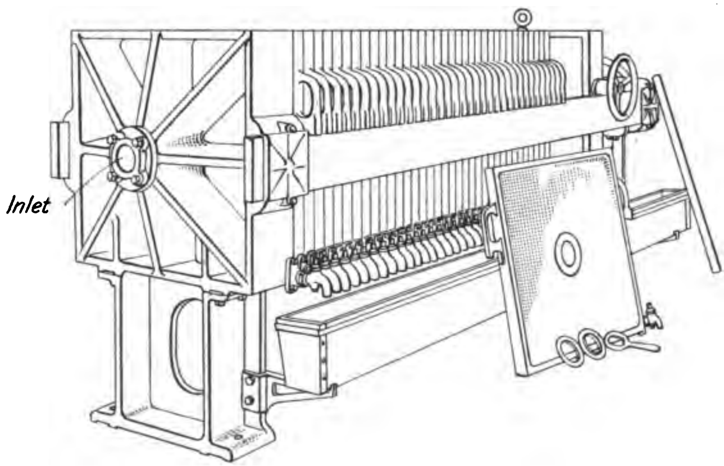


Fig. 66

ceases to flow from the outlet cocks, shown at the bottom of the cells, which indicates that it is time to shut off the pressure, open the press and remove the cakes. After the cakes have been removed, the press is closed again, and is then ready for the operation to be repeated.

It takes about 45 minutes to fill a press, and about 15 minutes to open, remove the cakes and close the press ready for another operation.

The consistency of sludge as it is forced into sludge presses is about 95 per cent. liquid and 5 per cent. solid. The pressed cakes contain about 70 per cent. moisture.

Filter presses are generally so installed that a dump cart can be backed under them, or they are placed over openings, or trap doors in the floor, so that when the presses are opened the cakes can drop into dump carts without any handling.

There is but very little odor to pressed sludge.

DISPOSAL OF EFFLUENTS

The point or place where the effluent from a purification works is to be discharged, determines the degree of purification required. For instance, when the effluent is to be discharged into tide water, into the lower reaches of a stream below the intake to a water supply system, or into large bodies of fresh water like the Great Lakes, the purification need be carried only to the point of reducing the easily putrescible matter, and the more slowly oxidized particles may be discharged into the water with the effluent. When effluent is to be discharged into good sized bodies of fresh water, or into streams having a dry weather flow of more than seven times the volume of sewage, the effluent as a rule need not be better than that from sprinkling filters and contact beds, after having been subjected to sedimentation to remove the flocculent matter carried in suspension. This would include the effluents from intermittent filters, sewage farms, and well-designed and managed septic tanks. Effluents which are to be discharged into small fresh water lakes or into streams having a dry weather flow of less than seven times the volume of sewage, should be able to pass the fish test, and effluents which are to be discharged into dry ditches or open channels leading to water courses should be capable of passing the incubation test.

DISPOSAL OF STORM WATER

There is no standard practice in the United States at present for the treatment of storm water, but in Great

Britain there are certain rules laid down by the Local Government Board, which must be observed. These rules require that sewage works, treating the sewage from separate systems, shall be large enough to care for twice the dry weather flow; and when treating sewage from combined sewer systems, three times the dry weather flow.

The law further requires additional capacity to treat storm water to the extent of four times the dry weather flow, in the case of separate sewer systems, and three times the dry weather flow in the case of combined sewer systems, at a maximum rate of 2,900,000 gallons per acre daily, so that in either case, or system of sewers, storm flows, up to six times the dry weather flow, must be treated. Any excess of storm water above this amount may be discharged directly into streams without treatment.

Storm water is not treated on the ordinary sewage beds, but special beds of coarse material are usually provided, which are operated continuously as long as the storm lasts, and allowed long periods of rest between. In order to keep the storm water beds in good condition, they are sometimes treated with small amounts of sewage between storms, care being taken that they are not clogged when needed for full duty.

The period of sedimentation is much reduced during storms, so that suspended matter is often carried to the storm beds, where, together with the grease which is sometimes present in large amounts, it may give rise to surface clogging. For this reason, storm beds require to be scraped at frequent intervals, or have the upper layers removed.

DESIGNING SEWAGE PURIFICATION PLANTS

GENERAL CONSIDERATION

Sewerage Systems—Sewage purification works are often designed for municipalities or communities where there are no existing sewerage systems, but where the sewerage system and purification works are to be designed together and with relation to each other. Generally, however, the purification plant is required to abate a nuisance caused by the discharge of crude sewage from the sewage outfalls, and must be designed to care for the sewage under the existing conditions. If the sewers are laid out on the separate system, so that the sanitary sewers receive the discharge of only the household wastes, together with what ground water infilters into the conduits, the problem is comparatively simple, as it necessitates only the collecting of the entire sewage from the various outfalls into an intercepting main, and extending the intercepting main to the site of the purification works. In some cases, extending the intercepting main to the site of the purification plant might necessitate the collection of sewage in sewage wells or sumps and pumping it to the purification works. This requirement would be the same, however, whether or not the sewerage system were constructed on the separate or combined system. When the sewers of the city are built on the separate system, the purification works need be made only large enough to care for a volume of sewage equal to the water supply of the city during the greatest consumption, together with the infiltering ground water, provision of course being made to increase the plant with increase of population. The volume of sewage to be cared for can be ascertained by gaugings, and no plant should be constructed without first determining, as nearly as possible, the exact flow of sewage and ground water to be cared for.

When the sewerage of a city is on the combined system, a separate system of sanitary sewers can be installed for the household waste, or, as is more common practice, the existing system of sewers can be utilized for both sewage and rain water, and provision made at the works to care for part of the water and allow the balance to overflow, without being purified after a certain percentage has been cared for. When this plan is resorted to, or when it is required, as it is in Great Britain, sufficient filter surface and tankage is provided to care for three times the dry weather flow of sewage on the regular filters, and the excess of this up to six times the dry weather flow, is treated on storm filters. All storm water over and above six times the dry weather flow is allowed to overflow to the sewer outfall.

When the sewerage system is to be built in connection with the purification works, the entire plant can usually be designed to work better and more economically than when the purification works are an after-consideration which must be adapted to existing conditions. As a rule, when designing a system of sewers in connection with a purification plant, the separate system will be found the more satisfactory, although local conditions might be such that the combined system would be preferable. The system best suited to the conditions can be determined only after a careful investigation of all circumstances bearing on the subject. In an arid region where the rainfall is light, and the cost of constructing separate storm sewers great, the better plan would doubtless be to use the combined system and make provision for treating the extra storm water, while in a rainy region, separate systems would probably be the more economical. From a strictly sanitary point of view, there is an advantage in having storm water treated at a purification works, as it thus insures the purification of all organic matter littering the streets before a rain storm. On the other hand, the increased volume of sewage to be cared for at the works, and at a time when the beds are already saturated with rain water, necessitates the purchase of such a large tract of land and the construction of such vast works as to be burdensome to most communities.

Quantity of Sewage to be Provided For—With the separate system of sewers, the quantity of sewage to be provided for will equal the volume of water consumed, plus what ground water infilters into the sewers.* The volume of ground water will depend to a great extent on the tightness of the joints in the sewer pipe, the porosity of the soil and the height of the water table. The only safe practice for determining the volume of sewage to be provided for is to gauge the flow of sewage through the main sewer, during wet weather and dry weather, at various hours during the twenty-four, and if possible, during the four seasons of the year, to determine the fluctuation in volume and the maximum flow. For country institutions, such as asylums, and hotels, where the sewage purification plant must be built at the time the buildings are erected, there are no means for determining the volume of sewage by gaugings, and in such cases an allowance of 100 gallons per capita per day will be found safe. Institutions of like character, now operating, use approximately 100 gallons of water per day per capita, and it is not likely that the consumption of water in other institutions to be built would exceed that limit. In designing the plant, however, it should be made large enough to care for the sewage from the largest number of inmates the buildings can accommodate.

In large cities where the houses are built closely together, and where the streets are paved so that all rain water from roofs, courts and streets will be carried off in storm sewers, there will be no surplus water to drain into the sewers and in such cases the volume of sewage will about equal the consumption of water.

In Table XVIII will be found the per capita daily consumption of water in the fifty largest cities in the United States in 1890 and 1900, arranged in order of population.

In cities where there are separate systems of sewers, rain leaders from the roofs of buildings are sometimes connected to the house drain. When this practice is followed, it adds considerably to the volume of sewage during rainstorms and should be provided for.

* At Grinnell, Iowa, the flow of sewage in wet weather is from three to four times the volume of water pumped from the city wells. No permanent water level, steepage at depths varying from 10 to 40 feet.

In cities having the separate system of sewers, in which the pipes are well joined with good tight joints, in the absence of exact data it may be assumed that the infiltration of ground water amounts to 15 per cent. If, however, the sewers are to be laid below the level of the water-table, pass through swampy ground, or follow the course of old covered streams, a greater allowance, perhaps 25 per cent., should be made.

A condition which it is as necessary to consider as the per capita consumption of water, is the fluctuation of

TABLE XVIII

Per capita daily water consumption in the fifty largest cities of the United States in 1890 and in 1900, arranged in order of population.*

Cities	Per Capita Consumption in Gallons		Increase or Decrease in Consumption in Ten Years in Gallons	
	1890	1900	Incr.	Decr.
1. †New York	79	116	37	....
2. Chicago	140	190	50	....
3. Philadelphia	132	229	97	....
4. †Brooklyn	72	....	....	....
5. St. Louis	72	159	87	....
6. Boston	80	143	63	....
7. Baltimore	94	97	3	....
8. San Francisco	61	73	12	....
9. Cincinnati	112	121	7	....
10. Cleveland	103	159	56	....
11. Buffalo	186	233	47	....
12. New Orleans	‡37	‡48	11	....
13. Pittsburgh	144	231	87	....
14. Washington	158	185	27	....
15. Detroit	161	146	....	15
16. Milwaukee	110	80	....	30
17. Newark	76	94	18	....
18. Minneapolis	75	93	18	....
19. Jersey City	97	160	63	....
20. Louisville	74	100	26	....
21. Omaha	94	176	82	....
22. Rochester	66	83	17	....
23. St. Paul	60	67	7	....

* The classification is by the census of 1890, so as to include all the cities in the earlier grouping.

† New York and Brooklyn consolidated since 1890.

‡ Only a small part of the population supplied.

TABLE XVIII—Continued

Cities	Per Capita Consumption in Gallons		Increase or Decrease in Consumption in Ten Years in Gallons	
	1890	1900	Incr.	Decr.
24. Kansas City	71	62	....	9
25. Providence	48	54	6	....
26. Denver	....	300	....	....
27. Indianapolis	71	79	8	....
28. Allegheny	230	....	....	....
29. Albany	....	191	....	....
30. Columbus	78	230	152	....
31. Syracuse	68	102	34	....
32. Worcester	59	70	11	....
33. Toledo	72	119	47	....
34. Richmond	167	100	....	67
35. New Haven	135	150	15	....
36. Paterson	128	129	1	....
37. Lowell	66	85	19	....
38. Nashville	146	140	....	6
39. Scranton	....	....	....	....
40. Fall River	29	36	7	....
41. Cambridge	64	79	15	....
42. Atlanta	36	84	48	....
43. Memphis	124	125	1	....
44. Wilmington	113	90	....	23
45. Dayton	47	62	15	....
46. Troy	125	183	58	....
47. Grand Rapids	....	156	....	....
48. Reading	75	92	17	....
49. Camden	131	280	149	....
50. Trenton	62	99.9	37.9	....

population. In some municipalities, for instance Coney Island, Atlantic City and Saratoga Springs, the winter population is only a fraction of what the visiting crowds swell the summer population to. When designing a purification plant for such a city the works should be built sufficiently elastic to care for the permanent population during the whole year, the maximum population during the summer, or any other number between these extreme limits. This may be accomplished by means of automatic or hand-operated weirs which may be raised or lowered, thus increasing or decreasing the capacity of the tanks, or which perhaps is the better method, by building the plants in a series of units, any one of which can be put in or thrown out of service as occasion demands.

In designing a system of sewers or a purification plant, the probable increase in population of the community is taken into consideration and provision made for the increase.

Law for Increase of Population—With very few exceptions, cities increase in population, and when designing a purification plant in order to provide for the increased volume of sewage, due to the increased population, it becomes necessary to determine at what rate the population will increase. If any unusual condition affects the city at the time the sewage purification works are projected, that is, if unusual opportunity for engaging in business, a boom in real estate, or anything that is likely to attract large numbers of people to that locality, the problem of determining what the population of the cities or villages will be at a future date becomes a matter more of conjecture than of fact. However, the population of cities which are experiencing only the normal growth of ordinary American cities can be fairly approximately forecasted. This can be done by constructing a ruled chart, and plotting on the chart the population for several past decades. For instance, take the population of the two cities, Lancaster, Pa., and Malden, Mass.; according to the census between 1800 and 1890, they had the population shown in Table XIX.

TABLE XIX—POPULATION OF LANCASTER, PA., AND MALDEN, MASS.

City	Year								
	1800	1810	1820	1840	1850	1860	1870	1880	1890
Lancaster, Pa.	4,292	5,405	7,704	8,417	12,369	17,603	20,233	25,769	32,011
Malden, Mass.	1,059	1,384	1,731	2,010	2,514	3,520	7,370	12,017	23,031

If these dates and populations be now plotted on a chart, as shown in Fig. 67, the diagram will show when the material increase in population commenced, and the uniformity or variation, as the case may be, between one decade and another. In the case of Lancaster, Pa., it will be seen that a marked increase in population commenced in 1840, and that there was an almost uniform increase of about 600 people per annum for the next fifty years. In that case, it would be safe to assume that the increase in population would continue at that rate for each 10,000

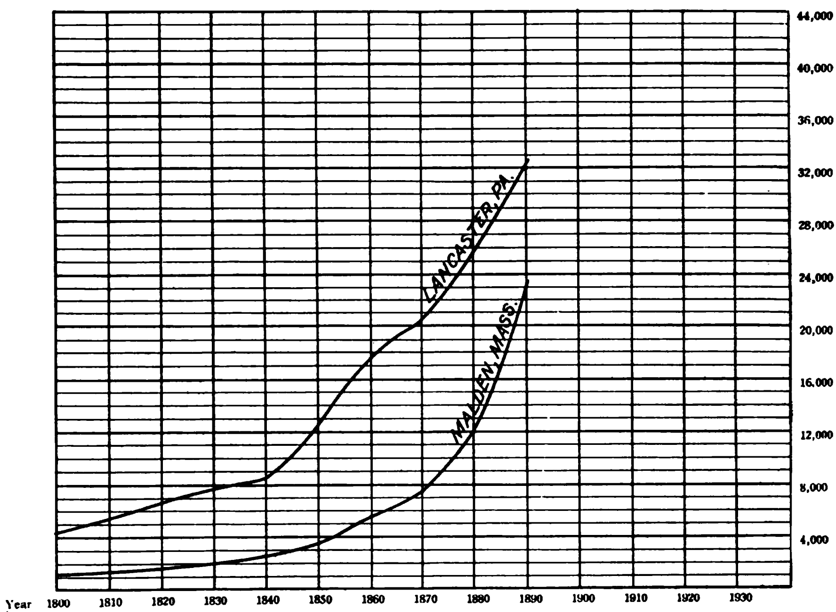


Fig. 67

inhabitants, and the population of the city at the end of twenty, thirty or forty years could be approximately foretold, providing no unusual conditions arose, either to increase or decrease the rate.

The growth of Malden, Mass., was much slower than that of Lancaster, but during the period from 1880 to 1890 the population of Malden increased approximately 11,000, as against about 6,500 for Lancaster. This establishes for Malden a rate of over 1,000 per year, which would probably represent the average increase for that city. Of course, some local cause, like the establishing or opening of mills or factories, might account for the increase in population in Malden during the decade from 1880 to 1890. In that case, that part of the population which was brought to the city by the unusual condition should be deducted from the increase in population for that period and the average increase then found for the twenty years from 1870 to 1890. In the absence of suitable data, it is sometimes assumed that American cities of less than 50,000 inhabitants increase in population at the rate of 50 per cent. in from eight to ten years, and that in cities of over 50,000 inhabitants the rate of increase is 50 per cent. in from sixteen to twenty years. This assumption, however, is of but little real value, as a reference to Table XX will show.

TABLE XX—POPULATION OF A NUMBER OF THE SMALLER CITIES AND TOWNS OF THE UNITED STATES AT EACH TEN YEAR PERIOD FROM 1800 TO 1900

Name of Cities	1800	1810	1820	1830	1840	1850	1860	1870	1880	1890
Alexandria, Va.	4,971	7,227	8,218	8,241	8,459	8,734	12,652	18,570	18,659	14,339
Akron, Ohio	....	....	....	....	....	3,266	3,477	10,006	16,512	27,600
Auburn, N. Y.	....	....	....	....	....	9,548	10,986	17,225	21,924	25,858
Augusta, Ga.	....	....	....	....	6,408	10,217	12,493	15,389	21,891	33,300
Bay City, Mich.	....	....	....	....	....	....	1,588	7,064	20,693	27,839
Burlington, Vt.	815	1,690	2,111	3,525	4,271	7,585	7,713	14,387	11,365	14,590
Binghamton, N. Y.	....	....	....	....	....	....	8,325	12,692	17,317	35,005
Chelsea, Mass.	....	....	....	....	2,390	6,701	13,395	18,547	21,882	27,909
Chester, Pa.	957	1,056	657	817	1,790	1,667	4,631	9,485	14,997	20,226
Cohoes, N. Y.	....	....	....	....	....	4,229	8,800	15.351	19,416	22,509
Dallas, Texas	....	....	....	....	....	....	....	....	10,358	38,067
Dover, N. H.	2,062	2,228	2,871	3,449	6,458	8,196	8,502	9,294	11,687	12,790
Danbury, Conn.	3,180	3,606	3,873	4,331	4,504	5,964	7,234	8,753	11,666	16,552
Fitchburg, Mass.	1,390	1,566	1,736	2,169	2,604	5,120	7,805	11,260	12,429	22,037
Hamilton, Ohio	....	....	....	....	....	3,210	7,233	11,081	12,122	17,565
Jacksonville, Fla.	....	....	....	....	....	1,045	2,118	6,912	7,650	17,201
Lancaster, Pa.	4,292	5,405	6,633	7,704	8,417	12,369	17,603	20,233	25,769	32,011
Malden, Mass.	1,059	1,384	1,731	2,010	2,514	3,520	5,865	7,370	12,017	23,031
Manchester, N. H.	....	....	761	877	3,235	13,932	20,017	23,536	32,630	44,126
Norristown, Pa.	....	....	827	1,089	2,937	6,024	8,848	10,753	13,063	19,791
Newport, Ky.	106	413	....	715	....	5,895	10,046	15,087	20,433	24,918
Steubenville, Ohio	....	....	2,539	2,937	4,247	6,140	6,154	8,107	12,093	13,394
San Antonio, Tex.	....	....	....	....	....	3,488	8,235	12,256	20,550	37,673
Wilkesbarre, Pa.	835	1,225	755	2,232	1,718	2,728	4,253	10,174	23,339	37,718
Williamsport, Pa.	131	334	624	....	1,353	1,615	5,664	16,030	18,934	27,132

Sewage Disposal in the United States—Raftor and Baker

Location for Purification Works—The best location for a purification works must be determined in each case after a careful examination of all available sites, and weighing the conditions in favor of each. When possible, it is well to locate the works within reasonable distance of the city, but if the cost of suitable property near the city outweighs the cost of pumping, it might be advisable to locate the plant at a considerable distance from the city, where sufficient land can be secured to provide for future growth of the city. The site for the purification works must be selected with a view to the final disposal of the effluent with the least trouble and expense. For this reason, if several sites offered about equal advantages, but one was so situated, as for instance on the lower reaches of a river below other settlements, that the effluent could be discharged into the water with a less degree of purification than at the other sites, the former location no doubt would be the best.

As a rule, the best location cannot be determined without a careful estimate of the cost of installation and maintenance of a plant built on the several sites.

Manufacturing Wastes—When the purification plant is to be designed for a manufacturing city where wastes from various manufacturing industries are discharged into the sewers, the problem of whether to require the various industrial wastes to be partly purified before discharging them into the sewers, or, assume that task as a community and design the plant accordingly, must be considered. Sometimes, at small cost, a manufacturing concern can so treat its sewage that it can safely be discharged into the sewers. In such cases, it is but reasonable to expect them to do so. In other cases the cost would be excessive, whereas the crude sewage, if discharged into the sewers, would become diluted and add but little to the cost of operating the municipal plant. In such cases, it would seem the part of wisdom to permit the discharge of crude sewage into the sewers. The only requirement in this regard when designing a plant, is that the policy to be provided for will be known to the engineer.

Degree of Purification Required—The degree of purification required will depend to a great extent on the place of disposal. It is seldom that an effluent of great purity is required, but simply one which can be discharged under the existing conditions without creating a nuisance. Such a requirement is complied with, even when the effluent contains considerable quantities of organic matter, provided the organic matter is in a fairly stable condition, so that it will not rapidly undergo putrefactive decomposition, or, if a sufficient amount of reserve oxygen is in the effluent or the water into which it is discharged, to unite with all readily oxidizable organic matter and thus prevent the development of anaërobic conditions.

Deciding on System of Purification—The system of sewage purification for any locality will generally be determined by the availability of the several materials in that locality, and other local conditions; while throughout the United States the practice will vary widely in different localities, within the various sections the practice will be found quite similar. For instance, in the New England States which are covered with a mantle of glacial drift that contains sand suitable for intermittent filtration, this method of purification no doubt will be extensively used. In the middle west, throughout the Mississippi River and Ohio River valleys, where suitable sand is more scarce, the septic tank and contact beds or sprinkling filters probably will find their greatest application; while in the arid regions of the far west where sewage is valuable for irrigation purposes, and the sparsely settled country offers suitable acres of land for farming, sewage irrigation will be the most suitable.

No hard and fast rules can be laid down to determine the type of disposal works to use under different conditions. The only way to decide is for the designer to familiarize himself with the various methods of purifying sewage, together with their advantages and limitations, and from the fullness of his knowledge and originality as a designer, after obtaining the necessary experimental data, decide upon the method for each case.

When Septic Tanks are Advisable—It is not an easy matter to state exactly when a septic tank is advisable. In the New England States, where suitable areas of sand of the right quality are available for intermittent filtration, septic tanks may be dispensed with and the crude sewage discharged directly onto the sand beds. If, however, the sand areas are limited so that higher rates of purification must be obtained, the septic tank will be found valuable for a preliminary treatment of the sewage. The septic tank will likewise be found valuable as a primary treatment in small towns or institutions which treat very fresh sewage, as the treatment breaks up the masses of fecal matter and other solids. It will also be found valuable in cities where there is a considerable quantity of manufacturing waste, as the tanks equalize the composition and the flow of sewage.

Beautifying Sewage Purification Plants—For sentimental, rather than for sanitary or economic reasons, a sewage purification plant should be made as attractive as possible; a judicious planting of trees and shrubbery, sodding of slopes and embankments, and painting of buildings so as to make the surroundings attractive, will go far toward overcoming the prejudice existing against the location of such works in a given neighborhood. If properly managed, there is but little or no odor from a purification plant, but that little odor will be exaggerated in the imagination of the people if the plant and surroundings are unattractive, while the odors will be entirely overlooked or attributed to other causes if the place is laid out as a parkway. Odoriferous plants, such as lilacs, will do much toward disguising any odors that do exist.

APPENDIX

Some Physical Properties of Sands and Gravels With Special Reference to Their Use in Filtration

By Allen Hazen

Chemist in charge of Lawrence Experiment Station

The experiments at the Lawrence Experiment Station under the direction of Hiram F. Mills, C. E., have necessitated many investigations in regard to the physical properties of filtering materials. The following is a brief account of some of the methods of analysis devised in the course of these investigations, together with the more important results obtained.

Method of Analysis

A knowledge of the sizes of the sand grains forms the basis of many of the computations. This information is obtained by means of mechanical analyses. The sand sample is separated into portions having grains of definite sizes, and from the weight of the several portions the relative quantities of grains of any size can be computed.

Collection of Samples—In shipping and handling, samples of sands are best kept in their natural moist condition, as there is then no tendency to separation into portions of unequal-sized grains. Under no circumstances should different materials be mixed in the same sample. If the material under examination is not homogeneous, samples of each grade should be taken in separate bottles, with proper notes in regard to location, quantity, etc. Eight-ounce wide-necked bottles are most convenient for sand samples, but with gravels a larger quantity is often required. Duplicate samples for comparison after obtaining the results of analyses are often useful.

Separation into Portions having Grains of Definite Sizes—Three methods are employed for particles of different sizes—hand picking for the stones, sieves for the sands and water elutriation for the extremely fine particles. Ignition, or determination of albuminoid ammonia, might be added for determining the quantity of organic matter, which, as a matter of convenience, is assumed to consist of particles less than 0.01 millimeter in diameter.

The method of hand picking is ordinarily applied only to particles which remain on a sieve two meshes to an inch. The stones of this size are spread out so that all are in sight, and a definite number of the largest are selected and weighed. The diameter is calculated from the average weight by the method to be described, while the percentage is reckoned from the total weight. Another set of the largest remaining stones is then picked out and weighed as before, and so on until the sample is exhausted. With a little practice the eye enables one to pick out the largest stones quite accurately.

With smaller particles this process becomes too laborious, on account of the large number of particles, and sieves are therefore used instead. The sand for sifting must be entirely free from moisture, and is ordinarily dried in an oven at a temperature somewhat above the boiling point. The quantity taken for analysis should rarely exceed 100–200 grams. The sieves are made from carefully selected brass-wire gauze, having as nearly as possible square and even-sized meshes. The frames are of metal, fitting into each other so that several sieves can be used at once without loss of material. It is a great convenience to have a mechanical shaker, which will take a series of sieves and give them a uniform and sufficient shaking in a short time; but without this good results can be obtained by hand shaking. A series which has proved very satisfactory has sieves with approximately 2, 4, 6, 10, 20, 40, 70, 100, 140 and 200 meshes to an inch; but the exact numbers are of no consequence, as the actual sizes of the particles are relied upon and not the number of meshes to an inch.

It can be easily shown by experiment that when a mixed sand is shaken upon a sieve the smaller particles pass first, and as the shaking is continued larger and larger particles pass, until the limit is reached when almost nothing will pass. The last and largest particles passing are collected and measured, and they represent the separation of that sieve. The size of separation of a sieve bears a tolerably definite relation to the size of the mesh, but the relation is not to be depended upon, owing to the irregularities in the meshes and also to the fact that the finer sieves are woven on a different pattern from the coarser ones, and the particles passing the finer sieves are somewhat larger in proportion to the mesh than is the case with coarser sieves. For these reasons the sizes of the sand grains are determined by actual measurements regardless of the size of the mesh and of the sieve.

It has not been found practicable to extend the sieve separations to particles below 0.10 millimeter in diameter (corresponding to a sieve with about 200 meshes to an inch), and for such particles elutriation is used. The portion passing the finest sieve contains the greater part of the organic matter of the sample, with the exception of roots and other large undecomposed matters, and it is usually best to remove this organic matter by ignition at the lowest possible heat before proceeding to the water separations. The loss in weight is regarded as organic matter, and calculated as below 0.01 millimeter in diameter. In case the mineral matter is decomposed by the necessary heat, the ignition must be omitted, and an approximate equivalent can be obtained by multiplying the albuminoid ammonia of the sample by 50*. In this case it is necessary to deduct an equivalent amount from the other fine portions, as otherwise the analyses when expressed in percentages would add up to more than one hundred.

Five grams of the ignited fine particles are put in a beaker 90 millimeters high, and holding about 230 cubic

*The method of making this determination was given in the *American Chemical Journal*, Vol. 12, p. 427.

centimeters. The beaker is then nearly filled with distilled water at a temperature of 20 degrees Centigrade, and thoroughly mixed by blowing into it air through a glass tube. A larger quantity of sand than 5 grams will not settle uniformly in the quantity of water given, but less can be used if desired. The rapidity of settlement depends upon the temperature of the water, so that it is quite important that no material variation in temperature should occur. The mixed sand and water is allowed to stand for fifteen seconds, when most of the supernatant liquid, carrying with it the greater part of the particles less than 0.08 millimeter, is rapidly decanted into a suitable vessel, and the remaining sand is again mixed with an equal amount of fresh water, which is again poured off after fifteen seconds, carrying with it most of the remaining fine particles. This process is once more repeated, after which the remaining sand is allowed to drain, and is then dried and weighed, and calculated as above 0.08 millimeter in diameter. The finer decanted sand will have sufficiently settled in a few minutes, and the coarser parts at the bottom are washed back into the beaker and treated with water exactly as before, except that one minute interval is now allowed for settling. The sand remaining is calculated as above 0.04 millimeter, and the portion below 0.04 is estimated by difference, as its direct determination is very tedious, and no more accurate than the estimation by difference when sufficient care is used.

Determination of the Sizes of the Sand Grains—The sizes of the sand grains can be determined in either of two ways, from the weight of the particles or from micrometer measurements. For convenience the size of each particle is considered to be the diameter of a sphere of equal volume. When the weight and specific gravity of a particle are known, the diameter can be readily calculated. The volume of a sphere is $\frac{1}{6}\pi d^3$, and is also equal to the weight divided by the specific gravity. With the Lawrence materials the specific gravity is uniformly 2.65 within very narrow limits, and we have $\frac{w}{2.65} = \frac{1}{6}\pi d^3$. Solving for d we obtain the formulæ $d = .9 \sqrt[3]{w}$ when d is the diameter of a particle in millimeters and w its weight in milligrams. As the average weight of particles, when not too small, can be determined with precision, this method is very accurate, and altogether the most satisfactory for particles above 0.10 millimeter ; that is, for all sieve separations. For the finer particles the method is inapplicable, on account of the vast number of particles to be counted in the smallest portion which can be accurately weighed, and in these cases the sizes are determined by micrometer measurements. As the sand grains are not spherical or even regular in shape, considerable care is required to ascertain the true mean diameter. The most accurate method is to measure the long diameter and the middle diameter at right angles to it, as seen by a microscope. The short diameter is obtained by a micrometer screw, focusing first upon the glass upon which the particle rests and then upon the highest point to be found. The mean diameter is then the cube root of the product of the three observed diameters. The middle diameter is usually about equal to the mean diameter, and can generally be used for it, avoiding the troublesome measurement of the short diameters.

The sizes of the separations of the sieves are always determined from the very last sand which passes through in the course of an analysis, and the results so obtained are quite accurate. With the elutriations average samples are inspected, and estimates made of the range in size of particles in each portion. Some stray particles both above and below the normal sizes are usually present, and even with the greatest care the result is only an approximation to the truth ; still, a series of results made in strictly the same way should be thoroughly satisfactory, notwithstanding possible moderate errors in the absolute sizes.

Calculation of Results—When a material has been separated into portions, each of which is accurately weighed and the range in the sizes of grains in each portion determined, the weight of the particles finer than each size of separation can be calculated and with enough properly selected separations the results can be plotted in the form of a diagram, and measurements of the curve taken for intermediate points with a fair degree of accuracy. This curve of results may be drawn upon a uniform scale using the actual figures of sizes and of per cents. by weight, or the logarithms of the figures may be used in one or both directions. The method of plotting is not of vital importance, and the method for any set of materials which gives the most easily and accurately drawn curves is to be preferred. In the diagram published last year the logarithmic scale was used in one direction, but in many instances the logarithmic scale can be used to advantage in both directions. With this method it has been found that the curve is often almost a straight line through the lower and most important section, and very accurate results are obtained even with a smaller number of separations.

Examples of Calculation of Results—Following are examples of representative analyses, showing the method of calculation used with the different methods of separation employed with various materials.

TABLE XXI—ANALYSIS OF A GRAVEL BY HAND PICKING, 11,870 GRAMS TAKEN FOR ANALYSIS

Number of Stones in Portion (Largest Selected Stones)	Total Weight of Portion Grams	Average Weight of Stones Milligrams	Estimated Weight of Smallest Stones Milligrams	Corresponding Size Millimeters	Total Weight of Stones Smaller than this Size	Per Cent. of Total Weight Smaller than this Size
. . .	.....	....	....	..	11,870	100
10 . .	3,320	332,000	250,000	56	8,550	72
10 . .	1,930	193,000	165,000	49	6,620	56
10 . .	1,380	138,000	124,000	45	5,240	44
20 . .	2,200	110,000	93,000	41	3,040	26
20 . .	1,520	76,000	64,000	36	1,520	13
20 . .	1,000	50,000	36,000	30	520	4.4
20 . .	460	23,000	10,000	20	60	.5
10 . .	40	4,000	2,000	11	20	.2
Dust .	20	....	....	..	......	......

The weight of the smallest stones in a portion given in the fourth column is estimated in general as about half-way between the average weight of all the stones in that portion and the average weight of the stones in the next finer portion.

The final results are shown by the figures in full-faced type in the last and third from the last columns. By plot-

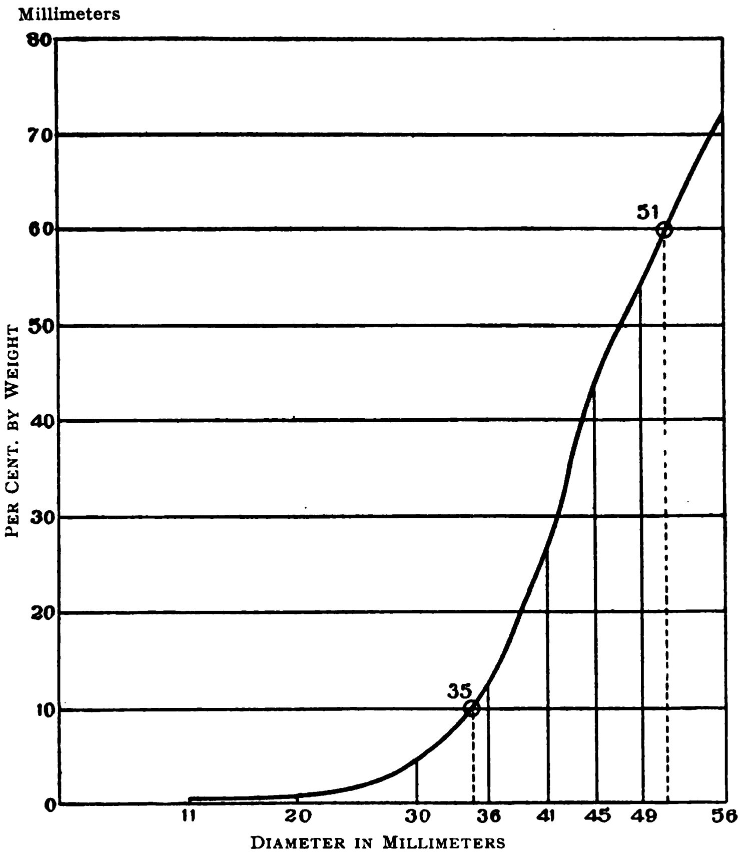


Fig. 68

ting these figures (Fig. 68) we find that 10 per cent. of the stones are less than 35 millimeters in diameter, and 60 per cent. are less than 51 millimeters. The uniformity coefficient, as described below, is the ratios of these numbers, or 1.46, while the "effective size" is 35 millimeters.

II. Analysis of a Sand by Means of Sieves

A portion of the sample was dried in a porcelain dish in an air bath. Weight dry, 110.9 grams. It was put into a series of sieves in a mechanical shaker, and given one hundred turns (equal to about seven hundred single shakes). The sieves were then taken apart, and the portion passing the finest sieve weighed. After noting the weight, the sand remaining on the finest sieve but passing all the coarser sieves was added to the first, and again weighed, this process being repeated until all the sample was upon the scale, weighing 110.7 grams, showing a loss by handling of only 0.2 grams. The figures were as follows:

Sieve Marked	Size of Separation of this Sieve Millimeters	Quantity of Sand Passing Grams	Per Cent. of Total Weight
190	**.105**	.5	**.5**
140	**.135**	1.3	**1.2**
100	**.182**	4.1	**3.7**
60	**.320**	23.2	**21.0**
40	**.46**	56.7	**51.2**
20	**.93**	89.1	**80.5**
10	**2.04**	104.6	**94.3**
6	**3.90**	110.7	**100.0**

Plotting the figures in heavy-faced type, we find from the curve (Fig. 69) that 10 and 60 per cent. respectively are finer than .25 and .62 millimeter, and we have for effective size, as described above, .25, and for uniformity coefficient 2.5.

III. Analysis of a Fine Material with Elutriation

The entire sample, 74 grams, was taken for analysis. The sieves used were not the same as those in the previous analysis, and instead of mixing the various portions on the scale they were separately weighed. The siftings were as follows:

Remaining on sieve	10, above 2.2	millimeters	1.5 grams.
Remaining on sieve	20, above .98	millimeters	7.0 grams.
Remaining on sieve	40, above .46	millimeters	22.0 grams.
Remaining on sieve	70, above .24	millimeters	20.2 grams.
Remaining on sieve	140, above .13	millimeters	9.2 grams.
Passing sieve	140, below .13	millimeters	14.1 grams.

The 14.1 grams passing the 140 sieve were thoroughly mixed, and one-third, 4.7 grams, taken for analysis. After ignition, just below a red heat in a radiator, the weight was diminished by 0.47 gram. The portion above .08 millimeter and between .04 and .08 millimeter, separated as described above, weighed respectively 1.27 and 1.71 grams, and the portion below .04 millimeter was estimated by difference

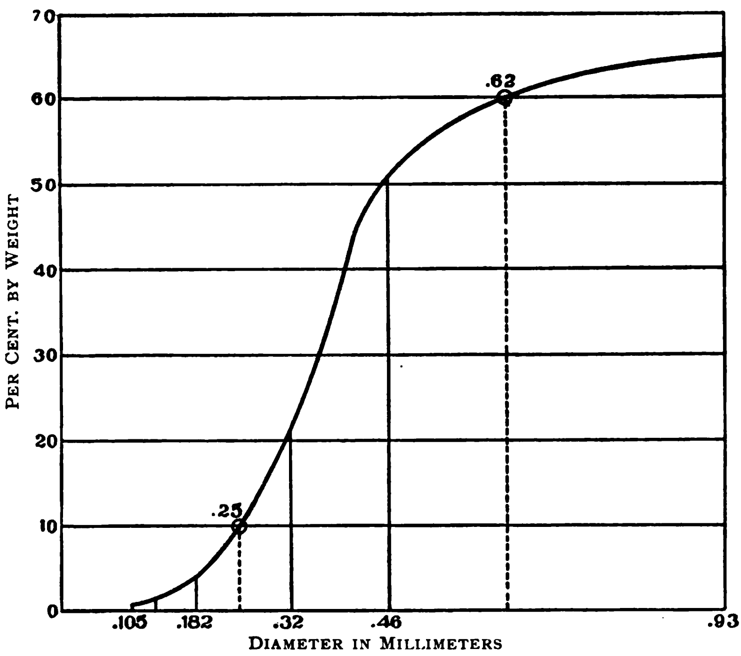


Fig. 69

(4 7 — (0.47 + 1.27 + 1.71)) to be 1.25 grams. Multiplying these quantities by 3, we obtain the corresponding quantities for the entire sample, and the calculation of quantities finer than the various sizes can be found on following page.

By plotting the heavy-faced figures, we find (Fig. 70) that 10 and 60 per cent. are respectively finer than .055 and .46 millimeter, and we have effective size .055 millimeter and uniformity coefficient 8.

Size of Grain	Weight Grams	Size of Largest Particles Millimeters	Weight of all the Finer Particles Grams	Per Cent. by Weight of all Finer Particles
Above 2.20	1.50	....	74.00	100
.98–2.20	7.00	2.20	72.50	98
.46– .98	22.00	.98	65.50	89
.24– .46	20.20	.46	43.50	60
.13– .24	9.20	.24	23.30	32
.08– .13	3.81	.13	14.10	19
.04– .08	5.13	.08	10.29	14
.01– .04	3.75	.04	5.16	7
Loss on ignition (assumed to be less than .01 millimeter)	1.41	.01	1.41	1.9

THE EFFECTIVE SIZE

As a provisional basis which best agrees with the known facts, the size of grain where the curve cuts the ten per cent. line is considered to be the "effective size" of the material. This size is such that 10 per cent. of the material is of smaller grains and 90 per cent. is of larger grains than the size given. The results obtained at Lawrence indicate that the finer 10 per cent. have as much influence upon the action of a material in filtration as the coarser 90 per cent. This is explained by the fact that in a mixed material containing particles of various sizes the water is forced to go around the larger particles and through the finer portions which occupy the intervening spaces, and so it is this finest portion which mainly determines the frictional resistance, the capillary attraction and, in fact, the action of the sand in almost every way.

Another important point in regard to a material is its degree of uniformity; whether the particles are mainly of the same size, or whether there is a great range in their diameters. This is conveniently shown by the "uniformity coefficient," a term used to designate the ratio of the size of grain which has 60 per cent. of the sample finer than itself to the size which has 10 per cent. finer than itself. These sizes are taken directly from the curve of results.

It is not probable that the above data regarding a sand include all the important points to be known, or that further study will not modify or change the method of calculation; but in the absence of better methods their use

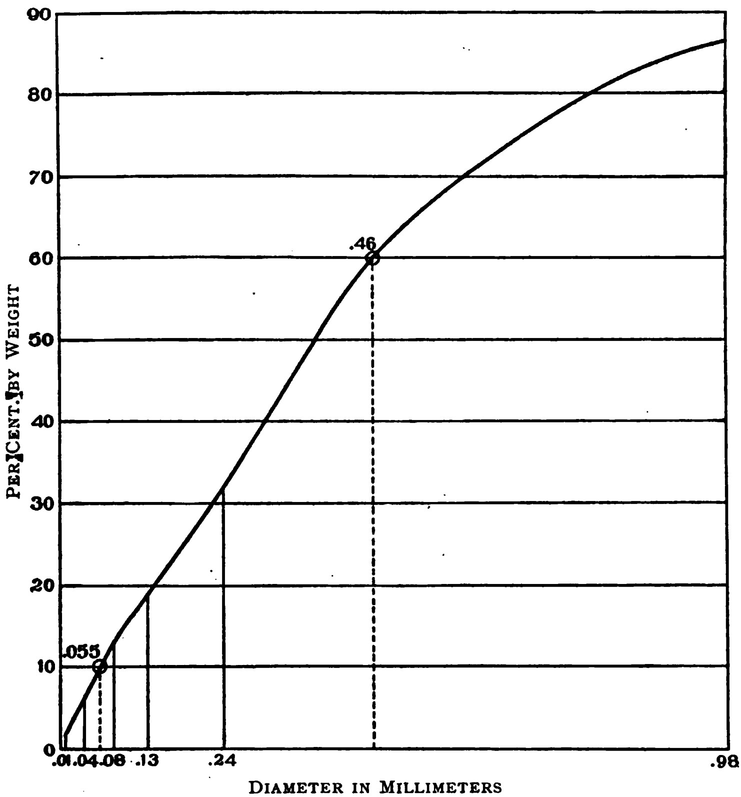


Fig. 70

allows extremely valuable approximate calculations, which would otherwise be almost impossible.

DETERMINATION OF OPEN SPACE AND WATER BY VOLUME

As it is often necessary to make determinations of open space and water in sands, a few notes in regard to the most suitable methods will be given.

The specific gravity of the solid particles is obtained by putting a weighed quantity of the thoroughly dry material into a narrow-necked graduated flask of distilled water, taking great care that no air bubbles are inclosed, and weighing the displaced water. Very accurate results may be obtained in this way. The specific gravity of the material as a whole is obtained by weighing a known volume packed as it is actually used, or as nearly so as possible. As the material is usually moist, it should either be dried before weighing or else a moisture determination made and a correction applied. The open space is invariably obtained by dividing the specific gravity of the material as a whole when dry by the specific gravity of the solid particles, and deducting the quotient from 1. The results obtained by measuring the quantity of water which can be put into a given volume when introduced from below are invariably too low, because the water is drawn ahead by capillarity, and air bubbles are enclosed and remain, often causing serious errors. A rough estimate of the open space can be made from the uniformity coefficient. Sharp-grained materials having uniformity coefficients below 2 have nearly 45 per cent. open space as ordinarily packed; and sands having coefficients below 3, as they occur in the banks or artificially settled in water, will usually have 40 per cent. open space. With more mixed materials the closeness of packing increases, until, with a uniformity coefficient of 6 to 8, only 30 per cent. open space is obtained, and with extremely high coefficients almost no open space is left. With round-grained water-worn sands the open space has been observed to be from 2 to 5 per cent. less than for corresponding sharp-grained sands.

The quantity of water contained in sand is obtained by drying a weighed portion in the usual way. The volume of the water is reckoned by the formula $V = sp.\ gr.\ \frac{M}{100-M}$ when *sp. gr.* is the specific gravity of the material as a whole when dry and M is the per cent. of moisture by weight. The difference between this figure and the open space is, in general, the air space.

CAPILLARITY

To determine the capillarity of a sand it is so placed that it is drained at a defined level, great care being taken to secure a compact packing free from stratification. Water is put freely upon it, and after a definite time, usually twenty-four or forty-eight hours, sand samples are taken at various levels and water determinations made as described above. The results plotted give a curve of "water capacity."*

The height to which water will be held to such an extent as to prevent the circulation of air can be roughly estimated by the formula $h=\frac{1.5}{d^2}$ when h is the height in millimeters and d the effective size of sand grain. The data from which the constant given above as 1.5 was calculated are very inadequate, and consequently the formula may require modification with more extended observations.

The height to which water is held by capillarity is independent of temperature.

DETERMINATION OF FRICTIONAL RESISTANCE

To determine the frictional resistance of a material, a cylinder of galvanized iron of convenient size is filled with the material packed under conditions as far as possible like those under which it is to be used. For water filtration the material is put loosely in position and settled to a compact condition by introducing water from below. Stratification must be carefully avoided. Water is then passed through at definite rates, keeping the material covered with an excess of water and regulating the rate of flow by the faucet at the bottom. The accompanying diagram (Fig. 71) represents a section of the apparatus (not drawn to scale). The loss of head between two points at a definite distance apart and both well within the material under examination is observed in glass tubes attached to pet cocks covered with fine wire gauze to keep back the

* The results of a number of such experiments were given in the annual report for 1891, page 432.

material. By proceeding in this way we eliminate the loss of head in the surface layer of sand, which is always much greater than for corresponding material below the surface, and is better studied by itself. The friction when the experiment is first started is always high, because many air bubbles are retained in the sand; but if water not

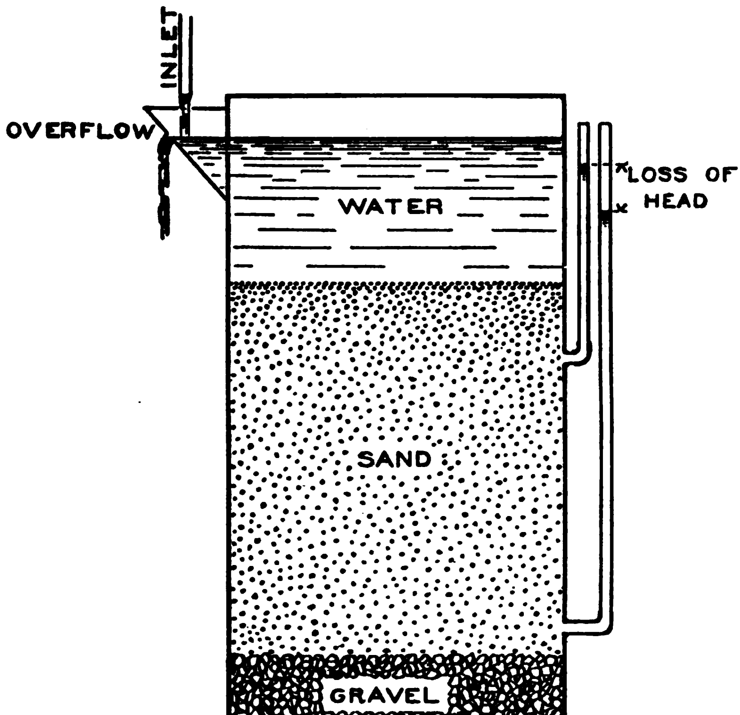


Fig. 71

entirely saturated with air is applied continuously for some days the air bubbles are absorbed and constant normal results are obtained.

FRICTION OF WATER IN SANDS AND GRAVELS

The frictional resistance of sand to water within certain limits of size of grain and rate of flow varies directly as the rate and as the depth of sand. This is given by Piefke* as Darcy's law. I have found that the friction also varies with the temperature, being twice as great at the freezing point as at summer heat both for coarse and fine sands, and also that with different sands the resistance varies inversely as the square of the effective size of the sand grain. It probably varies also somewhat with the uniformity coefficient, but no satisfactory data are at hand upon that point.

Putting the available data in the shape of a formula, we have

$$V = c\, d^2 \frac{h}{l}(.70 + .03t),$$

where

V is the velocity of the water in meters daily in a solid column of the same area as that of the sand,

c is a constant factor which present experiments indicate to be approximately 1,000,

d is the effective size of sand grain,

h is the loss of head,

l is the thickness of sand through which water passes,

t is the temperature on the Centigrade scale ($\frac{t \text{ Fahr.} + 10}{60}$ may be substituted for the last term, if desired).

The data at hand only justify the application of this formula to sands having a uniformity coefficient below 5, and effective size of grain 0.10 to 3.00 millimeters.

The quantity of water which will filter through a sand when its pores are completely filled with water and in the entire absence of clogging, with an active head equal to the depth of sand, and at a temperature of 10 degrees Centigrade, forms an extremely convenient basis for calculation, and for convenience is called the "maximum rate," as it is approximately equal to the greatest quantity of water which can be made to pass the sand under ordinary working conditions. Thus a sand with effective size, .20 millimeter, has a maximum rate of 40 meters per day; with effective

*Zeitschrift für Hygiene, Vol. VII, page 115.

size .30 millimeter, the maximum rate is 90 meters per day, etc.

TABLE XXII—SHOWING RATE AT WHICH WATER WILL PASS THROUGH DIFFERENT SANDS, WITH VARIOUS HEADS, AT A TEMPERATURE OF 10 DEGREES CENTIGRADE

$\frac{h}{l}$	Effective Size in Millimeters, 10 Per Cent. Finer than—						
	0.10	0.20	0.30	0.40	0.50	1.00	3.00
	Meters per Day	Meters per Day	Meters per Day	Meters per Day	Meters per Day	Meters per Day	Meters per Day
.001 . .	.01	.04	.09	.16	.25	1	9
.005 . .	.05	.20	.45	.80	1.25	5	45
.010 . .	.10	.40	.90	1.60	2.50	10	90
.050 . .	.50	2.	4.50	8.	12.50	50	..
.100 . .	1.	4.	9.	16.	25.	100	..
.500 . .	5.	20.	45.	80.	125.	...	..
1.000 . .	10.	40.	90.	160.	....	...	..
2.000 . .	20.	80.	180.	320.	....	...	..

The effect of variation in the temperature is shown by the following table:

RELATIVE QUANTITIES OF WATER PASSING AT DIFFERENT TEMPERATURES

Degrees, Centigrade	0	5	10	15	20	25	30
Degrees, Fahrenheit	32	41	50	59	68	77	86
Quantity	.70	.85	1.00	1.15	1.30	1.45	1.60

For gravels with effective sizes above 3 millimeters, the friction varies in such a way as to make the application of a general formula very difficult. As the size increases beyond this point, the velocity with a given head does not increase as rapidly as the square of the effective size; and with coarse gravels, the velocity varies as the square root of the head instead of directly with the head as in sands. The influence of temperature also becomes less marked with the coarse gravels.

The available data for materials above 3 millimeters, which are far less complete than could be desired, have been obtained entirely from screened gravels with uniformity coefficients from 1.4 to 2.0, and at a temperature of 10 degrees C., or a little above. The results obtained were plotted, making a diagram from which the table below has been prepared. The figures given in the table must be taken as provisional, and for use only until more extended results are obtained.

TABLE XXIII—SHOWING RATE AT WHICH WATER WILL PASS THROUGH DIFFERENT GRAVELS WITH VARIOUS HEADS

$\frac{h}{l}$	Effective Size in Millimeters, 10 Per Cent. Finer than—				
	3	5	8	10	15
	Meters per Day	Meters per Day	Meters per Day	Meters per Day	Meters per Day
.0005	3.5	10	20	30	50
.001	7	21	41	58	100
.002	14	40	78	110	190
.004	27	77	150	208	350
.006	41	112	207	275	450
.008	54	142	252	340	530
.010	67	173	300	385	610
.015	98	238	373	480	760
.020	127	300	467	580	890
.030	185	400	615	750	1,110
.050	280	560	885	1,060	1,490
.100	495	930	1,310	1,550	......

$\frac{h}{l}$	Effective Size in Millimeters, 10 Per Cent. Finer than—				
	20	25	30	35	40
	Meters per Day	Meters per Day	Meters per Day	Meters per Day	Meter per Day
.0005	80	110	150	200	250
.001	148	205	275	370	450
.002	275	370	480	590	710
.004	480	610	740	870	1,000
.006	620	780	930	1,090	1,240
.008	720	900	1,090	1,270	1,450
.010	830	1,030	1,220	1,410	......
.015	1,030	1,260	1,480	......	......
020	1,180	1,470	......	......	......
.030	1,450	......	......	......	......
.050	......	......	.....	......	......
.100	......	......	.....	. ....	......

In making calculations in regard to underdrains for either sewage of water filters, or in regard to the movements of ground waters, there should be no perceptible clogging of porous materials free from stratification by a clear ground water, and the formulæ given can be used with only a moderate factor of safety to cover possible errors of sampling, analysis, and errors in the formulæ themselves. In estimating the actual capacity of a filter, so many other conditions come in—the presence of air bubbles and especially the increased friction in the upper layers—that it is impossible to calculate the practicable rate of flow by formulæ, and we can only safely rely upon actual results from known materials.

The analyses of the materials used at Lawrence have been given in previous reports of the board in connection with the results obtained from them. The following table contains the result of analyses of some other materials, which may be of general interest:

TABLE XXIV—MECHANICAL ANALYSES OF SANDS.

	Effective Size 10 Per Cent. Finer than—Millimeters	Uniformity Coefficient
Filter Tank No. 1, Lawrence, Mass. . . .	.48	2.4
Filter Tank No. 9, Lawrence, Mass. . . .	.18	2.0
Filter Tank No. 2, Lawrence, Mass. . . .	.08	2.0
Sewage filters, Gardner, Mass.	.10–.24	6.14
Sewage filters, Marlborough, Mass. . . .	.12	3.4
Sewage filters, South Framingham, Mass. .	.35–.42	4.5
Water filter, Lawrence, Mass.	.25–.30	2.5– 4.5
Water filter, Birmingham, Eng.	.27	1.8
Water filter, Southwalk & Vauxhall Co., London, Eng.	.29	2.0
Water filter, Poughkeepsie, N. Y.	.25–.35	1.8– 1.9

The data already collected clearly show that a well-selected material is essential to successful filtration; and, with the method of examination and calculation now proposed, we can decide with confidence many otherwise indefinite points, and thus avoid unnecessary expense and unsatisfactory results from the use of unsuitable or poorly arranged materials.

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